ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG, MS

(6) CORPS OF ENGINEERS

HYDRAULIC DESIGN CRITERIA. Volume 2.

[Revised January 1977]

(1)

AD A092238 VOLUME 2.

LEVEL III

PREFACE

The purpose of Volume 2 of "Hydraulic Design Criteria" is to prevent overcrowding of Volume 1 and to facilitate use of the design charts. To accomplish this purpose it will be necessary to divide Hydraulic Design Criteria from time to time as the number of charts increases. The revised tables of contents included with each new issue of Hydraulic Design Criteria will divide the charts in an appropriate manner.

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CORPS OF ENGINEERS

HYDRAULIC DESIGN CRITERIA

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HYDRAULIC DESIGN CRITERIA

SHEETS 310-1 TO 310-1/2

WAVE PRESSURES ON CREST GATES

1. A theory for the pressure resulting from a wave striking a vertical wall was developed by Sainflou (1). The particular phenomenon is known as a "clapotis." The incident wave combines with the reflected wave to produce a wave height twice that of the incident wave. The theory is valid only for wave heights which do not exceed the still-water depth. The depth of water behind spillway crest gates is normally greater than the design wave height. Therefore, the theory can be used to estimate pressure distribution for the design of crest gates and for spillway stability analysis problems.

2. Application of the Sainflou wave pressure theory to crest gates and spillways is illustrated on Hydraulic Design Chart 310-1. The first equation is a parameter of the clapotis and indicates the effective change in mean water depth resulting from transition of the wave. The second equation indicates the change in bottom pressure. The clapotis results in pressure decrease as well as a pressure increase relative to the still-water static pressure. Design problems are generally only concerned with the maximum pressure.

3. Overtopping of a gate by waves occurs when the clapotis rises above the gate. For this condition the maximum pressure distribution would be zero at the top of the gate and vary along a curve which would become asymptotic to the straight-line distribution at the bottom of the spillway structure. As data are not available to establish the true pressure distribution, it may be assumed for design purposes that the portion of the pressure diagram above the top of the gate is ineffective and that the pressure distribution below the top of the gate is a straight line as indicated on Chart 310-1.

4. The equations of the clapotis involve hyperbolic functions of the cosine and cotangent. Hydraulic Design Chart 310-1/1 presents graphical and tabulated values of these functions for depth-wave length ratios (D/λ) of 0.0 to 0.8.

5. Hydraulic Design Chart 310-1/2 is a sample computation illustrating use of the Sainflou theory for crest gate design and spillway stability analysis. A wave length, wave height, and approach depth of 125, 6, and 75 ft, respectively, have been assumed for the computation. The direction of approach is considered normal to the spillway.

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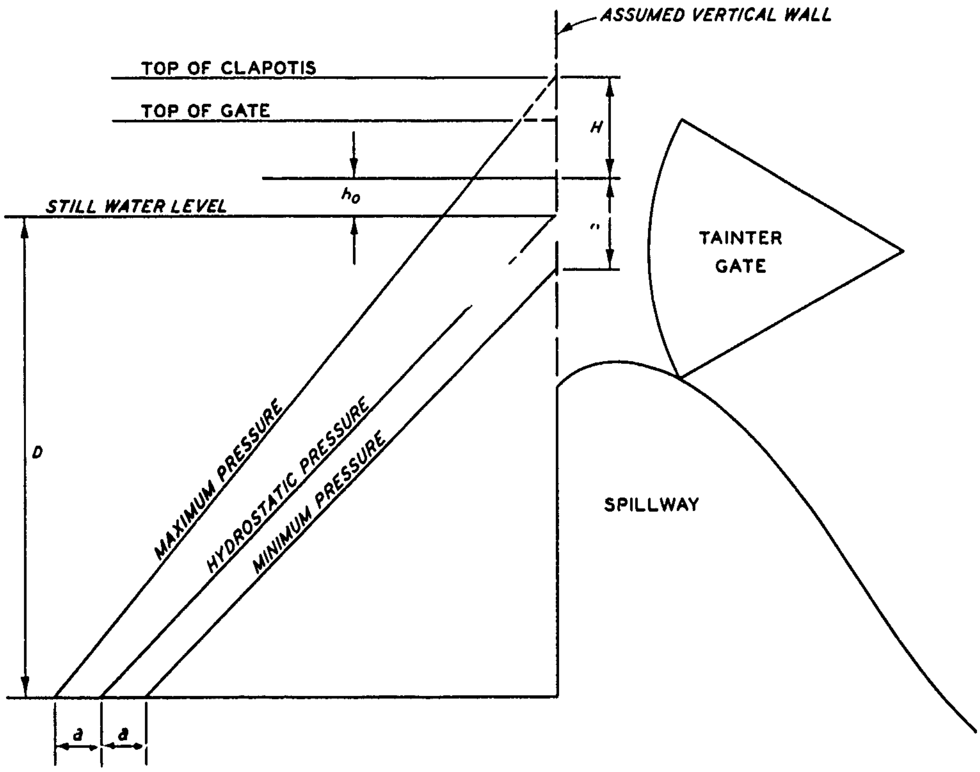
(1) M. Sainflou, "Essay on vertical breakwaters," Annales des Ponts et Chaussees (July-August 1928), pp 5-48. Translated by C. R. Hatch for U. S. Army Engineer Division, Great Lakes, CE, Chicago, Ill. (No date.)



EQUATIONS

$$h_o = \frac{\pi H^2}{\lambda} \coth \frac{2\pi D}{\lambda}$$

$$a = \frac{H}{\cosh \frac{2\pi D}{\lambda}}$$

WHERE

- h_o = A PARAMETER OF THE CLAPOTIS, FT
- a = A BOTTOM PRESSURE PARAMETER, FT OF WATER
- D = DEPTH OF WATER (STILL WATER LEVEL TO BOTTOM), FT
- H = WAVE HEIGHT, FT
- λ = WAVE LENGTH, FT

NOTE VALUES OF $\cosh \frac{2\pi D}{\lambda}$ AND $\coth \frac{2\pi D}{\lambda}$ ARE ON CHART 310-1/1

CREST GATES

WAVE PRESSURE DESIGN ASSUMPTIONS

HYDRAULIC DESIGN CHART 310-1

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

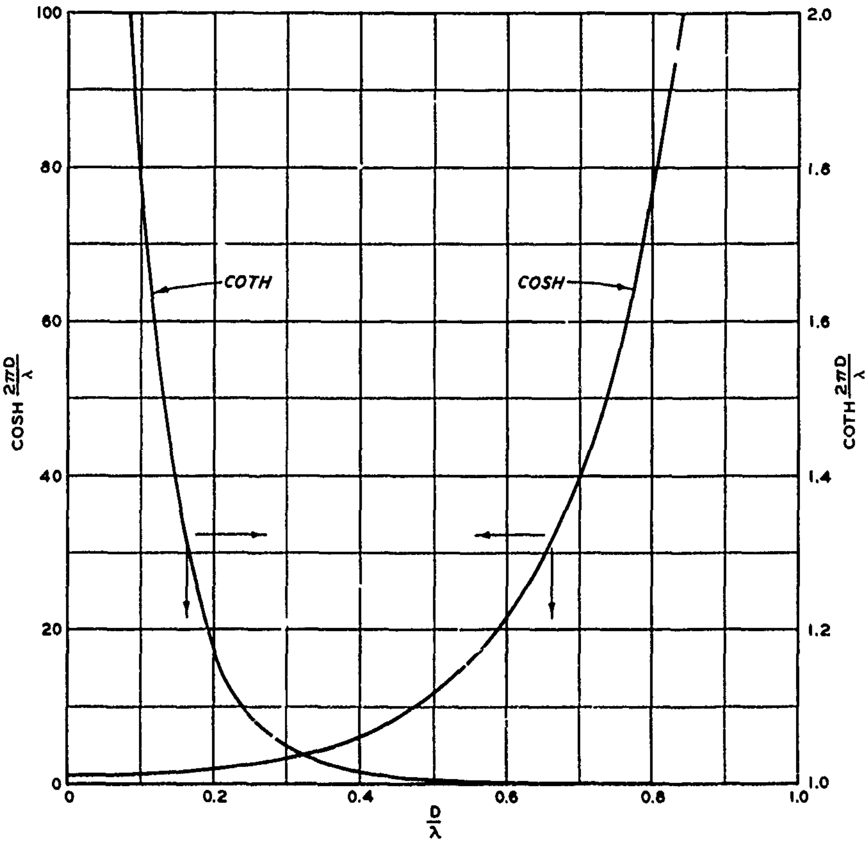


TABLE OF VALUES

$\frac{D}{\lambda}$	COSH $\frac{2\pi D}{\lambda}$	COTH $\frac{2\pi D}{\lambda}$
0	1.000	∞
0.1	1.204	1.796
0.2	1.898	1.177
0.3	3.366	1.047
0.4	6.205	1.013
0.5	11.574	1.004
0.6	21.659	1.001
0.7	40.569	1.000
0.8	76.013	1.000

NOTE: D=DEPTH OF WATER (STILL WATER LEVEL TO BOTTOM), FT
λ= WAVE LENGTH, FT

CREST GATES

WAVE PRESSURE
HYPERBOLIC FUNCTIONS

HYDRAULIC DESIGN CHART 310-1/1

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

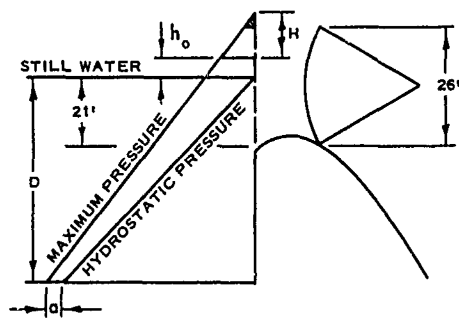
JOB CW 804 PROJECT John Doe Dam SUBJECT Crest Gates

COMPUTATION Effects of Wave Pressure

COMPUTED BY RGC DATE 6/3/60 CHECKED BY MBB DATE 6/7/60

GIVEN:

Gated spillway as shown
Design wave length (λ) = 125 ft
Design wave height (H) = 6 ft
Still-water depth (D) = 75 ft



REQUIRED:

1. Maximum pressure distribution on gate and spillway structure
2. Maximum hydraulic load per ft of width of gate
3. Maximum hydraulic load per ft of width of structure

COMPUTE:

1. Pressure distribution

(a) Maximum effective depth with wave

$$h_o = \frac{\pi H^2}{\lambda} \coth \frac{2\pi D}{\lambda} \quad \text{(Chart 310-1)}$$

$$\frac{D}{\lambda} = \frac{75}{125} = 0.6; \ \coth \frac{2\pi D}{\lambda} = 1.001 \quad \text{(Chart 310-1/1)}$$

$$h_o = \frac{3.14 \times 6^2}{125} \times 1.001 = 0.9 \text{ ft.}$$

Effective depth = $D + h_o + H = 75.0 + 0.9 + 6.0 = 81.9$ ft.

(b) Maximum effective bottom pressure with wave

$$a = \frac{H}{\cosh \frac{2\pi D}{\lambda}} \quad \text{(Chart 310-1)}$$

$$\frac{D}{\lambda} = 0.6; \ \cosh \frac{2\pi D}{\lambda} = 21.7 \quad \text{(Chart 310-1/1)}$$

$$a = \frac{6}{21.7} = 0.3 \text{ ft.}$$

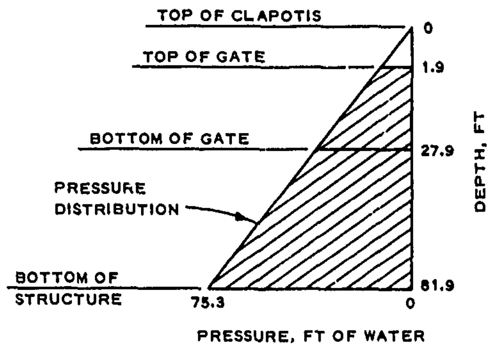
Effective pressure = $D + a = 75.0 + 0.3 = 75.3$ ft.

CREST GATES
WAVE PRESSURE
SAMPLE COMPUTATION
HYDRAULIC DESIGN CHART 310-1/2

(c) Depth of gate overtopping

Depth = 81.9 – (75.0 – 21.0 + 26.0) = 1.9 ft.

(d) Maximum pressure distribution graph



2. Maximum hydraulic load per foot of width of gate (from 1d above)

Maximum pressure at top of gate $(P_1) = \frac{1.9}{81.9} \times 75.3 = 1.7$ ft

Maximum pressure at bottom of gate $(P_2) = \frac{27.9}{81.9} \times 75.3 = 25.7$ ft

Maximum hydraulic load on gate $(R) = \gamma \left(\frac{P_1 + P_2}{2}\right) \times$ gate height

γ = specific weight of water = 62.4 lb/ft^3

$$R = 62.4 \left(\frac{1.7 + 25.7}{2}\right) 26$$

$$= 22{,}200 \text{ lb/ft of width}$$

Note: For still-water level maximum gate pressure is 21 ft of water and maximum hydraulic load is 13,750 lb/ft of width.

3. Maximum hydraulic load per foot of width of structure (from 1d above)

Maximum pressure at bottom of structure $(P_3) = 75.3$ ft

Maximum hydraulic load on structure $(R_h) = \gamma \left(\frac{P_1 + P_3}{2}\right) \times$ height of structure

$$= 62.4 \left(\frac{1.7 + 75.3}{2}\right) 80$$

$$= 192{,}000 \text{ lb/ft of width}$$

Note: Equivalent for still-water level is 175,000 lb/ft of width.

CREST GATES

WAVE PRESSURE

SAMPLE COMPUTATION

HYDRAULIC DESIGN CHART 310-1/2

HYDRAULIC DESIGN CRITERIA

SHEETS 311-1 TO 311-5

TAINTER GATES ON SPILLWAY CRESTS

DISCHARGE COEFFICIENTS

1. Discharge through a partially open tainter gate mounted on a spillway crest can be computed using the basic orifice equation:

$$Q = CA\sqrt{2gH}$$

where,

Q = discharge in cfs

C = discharge coefficient

A = area of orifice opening in ft^2

H = head to the center of the orifice in ft.

The coefficient (C) in the above equation is primarily dependent upon the characteristics of the flow lines approaching and leaving the orifice. In turn, these flow lines are dependent upon the shape of the crest, the radius of the gate, and the location of the trunnion.

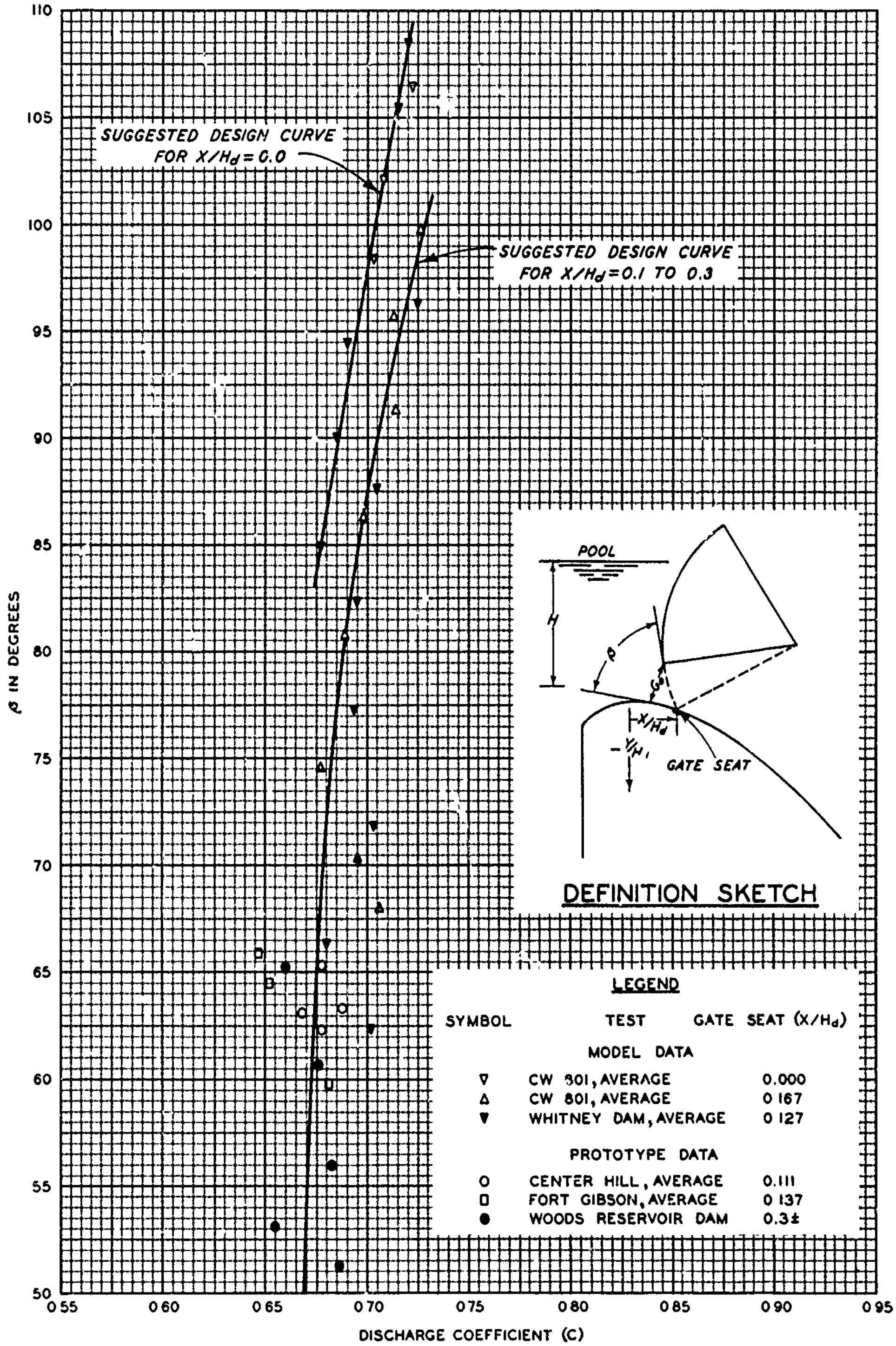
2. Discharge Coefficients. Chart 311-1 shows a plot of average discharge coefficients computed from model and prototype data for several crest shapes and tainter gate designs for nonsubmerged flow. Data shown are based principally on tests with three or more bays in operation. Discharge coefficients for a single bay would be lower because of side contractions although data are not presently available to evaluate this factor. On this chart, the discharge coefficient (C) is plotted as a function of the angle (β) formed by the tangent to the gate lip and the tangent to the crest curve at the nearest point of the crest curve. The net gate opening is considered to be the shortest distance from the gate lip to the crest curve. The angle is a function of the major geometric factors affecting the flow lines of the orifice discharge. One suggested design curve applies to tainter gates having gate seats located downstream from the crest axis. The other suggested design curve is based on tests with the gate seat located on the axis and indicates the effects of the masonry shape upstream from the crest axis.

3. Computation. Computation of discharge through a tainter gate mounted on a spillway crest is considerably complicated by the geometry involved in determining the net gate opening to be used in the orifice formula. The problem is simplified by fitting circular arcs to the crest

curve used in the design of spillways. Chart 311-2 illustrates the necessary computations to obtain the net gate opening and the angle β described in paragraph 2, for tainter gates mounted on spillway crests shaped to $X^{1.85} = -2\,H_d^{0.85}Y$. All factors are expressed in terms of the design head (H_d). The method shown is applicable to other crest shapes. However, the accompanying design aids, Charts 311-3 and 311-4, apply only to standard crests.

4. To initiate the computations, Y_L/H_d values of the gate lip are assumed and corresponding values of X_L/H_d are computed (columns 1 to 6, Chart 311-2). These coordinates are then located on Chart 311-3 to determine the characteristics of a substitute arc. The substitute arc is then used to compute the net gate opening (columns 7 to 14). The point of intersection of the masonry line by the gate opening is determined by similar triangles (columns 14, 15, and 16). Design aid Chart 311-4 can be used to determine the Y_c/H_d coordinate of the gate opening and masonry line intersection (column 17), and also the slope of the masonry line (columns 18 and 19) which in turn combines with the slope of the gate lip tangent to form the angle β (column 20). If graphical methods are preferred to analytical methods, a large-scale layout will enable the head, net gate opening, and the angle β to be scaled so that the discharge can be computed with fair accuracy.

5. Chart 311-5 is a sample computation of the steps involved in the development of a rating curve for a partially open tainter gate. The final computations are dimensional and are believed accurate to within ± 2 per cent, for gate opening-head ratios (G_o/H) less than 0.6.



FORMULA

$$Q = C G_o B \sqrt{2gH}$$

WHERE

G_o = NET GATE OPENING
B = GATE WIDTH
H = HEAD TO CENTER OF GATE OPENING

TAINTER GATES ON SPILLWAY CRESTS

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 311-1

WES 3-56

COMPUTATION SHEET

GATE OPENINGS AND ANGLE β

JOB CW804 PROJECT JOHN DOE DAM
SUBJECT SPILLWAY DISCHARGE
COMPUTED BY AAMC DATE 8-24-54
CHECKED BY HAB DATE 8-26-54

GIVEN

DESIGN HEAD (H_d) = 37.0 FT
RADIUS OF GATE (R_G) = $0.831 H_d$
TRUNNION COORDINATES (X_T, Y_T)
$X_T = 0.907 H_d$, $Y_T = 0.324 H_d$.

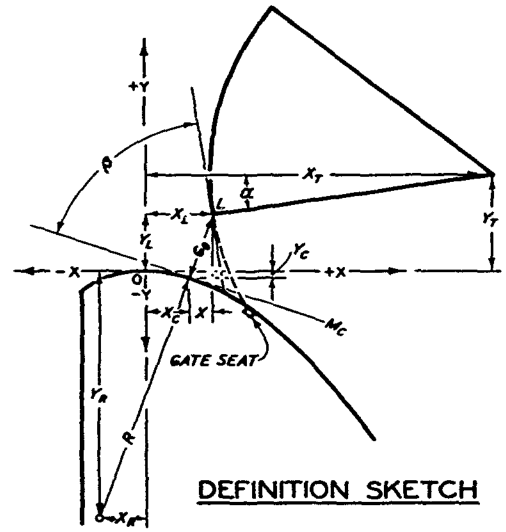
DEFINITIONS

GATE LIP COORDINATES (X_L, Y_L).
SPILLWAY CREST COORDINATES (X_C, Y_C).
SLOPE OF TANGENT TO CREST (M_C), NEGATIVE WHEN DOWNSTREAM FROM CREST.
SHORTEST DISTANCE FROM GATE LIP TO CREST (G_0).

DEFINITIONS (CONT)

α IS THE ANGLE BETWEEN A LINE CONNECTING THE GATE LIP AND THE TRUNNION CENTER, AND A HORIZONTAL LINE THROUGH THE TRUNNION, CONSIDERED POSITIVE AND NEGATIVE WHEN THE GATE LIP IS ABOVE AND BELOW THE TRUNNION, RESPECTIVELY.

NOTE: ALL DIMENSIONS USED IN COMPUTATIONS ARE IN TERMS OF DESIGN HEAD (H_d).



DEFINITION SKETCH

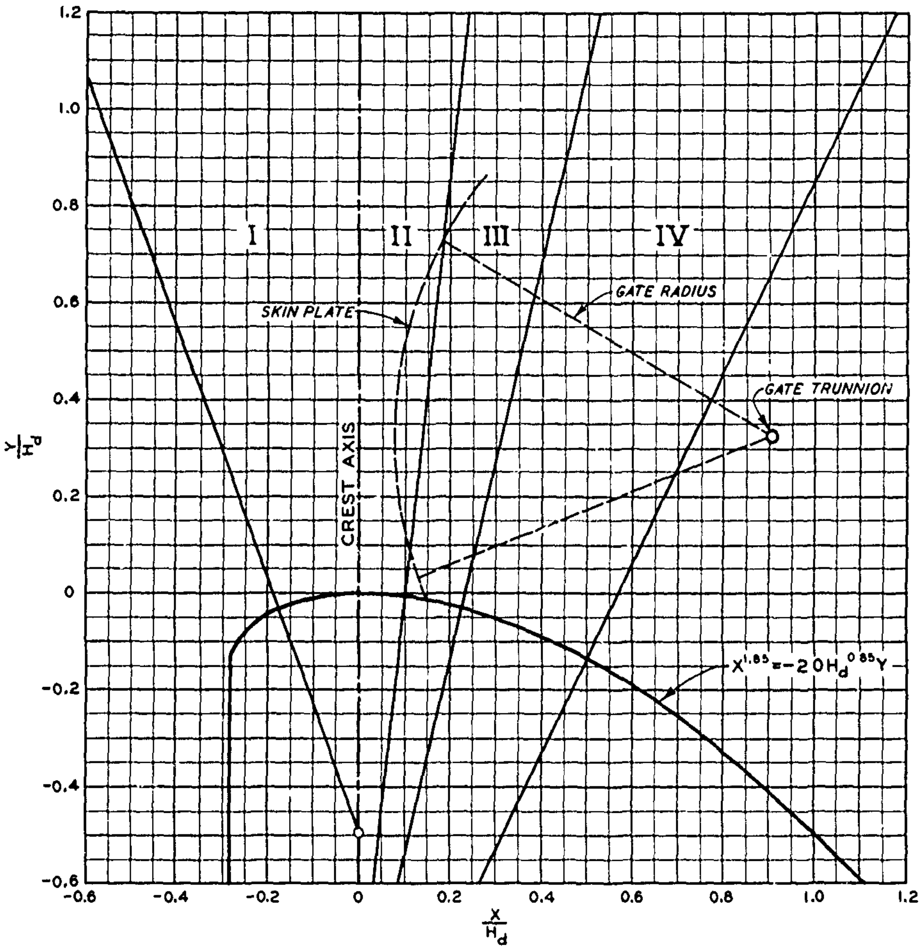
(1) Y_L	(2) Y_T-Y_L	(3) SIN α (2) ÷ R_G	(4) α DEGREES	(5) R_GCOSα 0.831COSα	(6) X_L X_T−(5)	(7) FROM CHART 311-3 CLASS	(8) X_R	(9) Y_R	(10) R	(11) X_L-X_R (6)−(8)	(12) Y_L-Y_R (1)−(9)	(13) $R+G_0$ $[(11)^2+(12)^2]^{1/2}$	(14) G_0 (13)−(10)	(15) X (11) X (14) / (13)	(16) X_C (6)−(15)	(17) FROM CHART 311-4 Y_C	(18) M_C	(19) $TAN^{-1}M_C$ DEGREES	(20) β $90°+TAN^{-1}M_C+\alpha$ 90°+(19)+(4) DEGREES
0.100	0.224	0.270	−15.67	0.800	0.107	II	−0.050	−1.329	1.330	0.157	1.429	1.437	0.107	0.012	0.095	−0.0065	−0.125	−7.13	67.20
0.200	0.124	0.149	−8.57	0.821	0.086	II	−0.050	−1.329	1.330	0.136	1.529	1.535	0.205	0.018	0.068	−0.0035	−0.094	−5.35	76.08
0.300	0.024	0.029	−1.66	0.830	0.077	II	−0.050	−1.329	1.330	0.127	1.629	1.634	0.304	0.024	0.053	−0.0022	−0.076	−4.36	83.98
0.400	−0.076	−0.091	+5.22	0.829	0.078	II	−0.050	−1.329	1.330	0.128	1.729	1.733	0.403	0.030	0.048	−0.0018	−0.070	−4.02	91.20

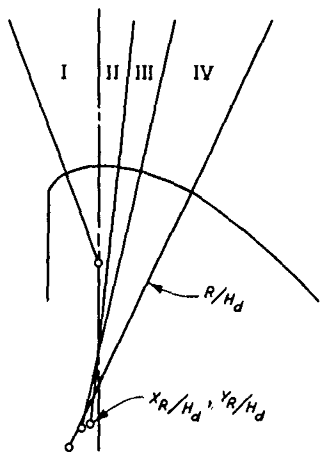
TAINTER GATES ON SPILLWAY CRESTS

SAMPLE GEOMETRIC COMPUTATION

HYDRAULIC DESIGN CHART 311-2

WES 3-56





DEFINITION SKETCH

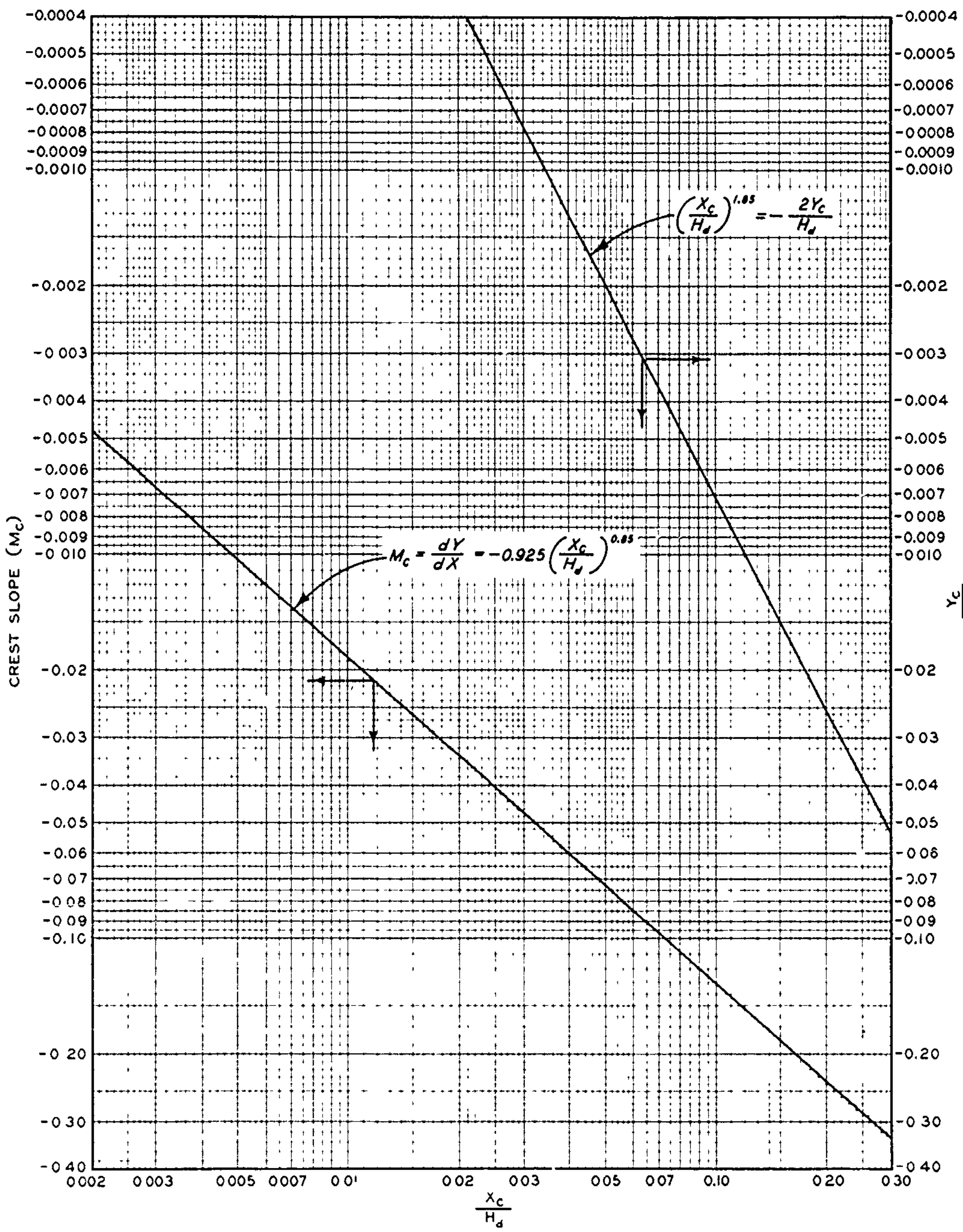
CLASS	R/H_d	X_R/H_d	Y_R/H_d
I	0.500	0.000	-0.500
II	1.330	-0.050	-1.329
III	1.359	-0.100	-1.351
IV	1.472	-0.164	-1.452

TAINTER GATES ON SPILLWAY CRESTS

GEOMETRIC FACTORS

HYDRAULIC DESIGN CHART 311-3

WES 3-56



TAINTER GATES ON SPILLWAY CRESTS

CREST COORDINATES AND SLOPE FUNCTION

HYDRAULIC DESIGN CHART 311-4

WES 3-56

WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB CW804 PROJECT JOHN DOE DAM SUBJECT SPILLWAY DISCHARGE

COMPUTATIONS COORDINATES FOR RATING CURVE (POOL VS DISCHARGE FOR VARIOUS GATE OPENINGS)

COMPUTED BY AAMC DATE 8-25-54 CHECKED BY RRW DATE 8-27-54

GIVEN

DESIGN HEAD (H_d) = 37.0 FT
GATE WIDTH (B) = 42.0 FT
CREST ELEV = 288.0 FT

FORMULAS

$Q = CG_0B\sqrt{2gH}$
H = POOL ELEV − 0.5[ELEV Y_L + ELEV Y_C]

(1) β* DEGREES	(2) C**	(3) G_0/H_d*	(4) G_0 FT	(5) Y_L/H_d*	(6) Y_L FT	(7) Y_C/H_d*	(8) Y_C FT	(9) ELEV Y_L = 288 + Y_L FT	(10) ELEV Y_C = 288 + Y_C FT	(11) $\frac{(9)+(10)}{2}$	(12) POOL FT	(13) H (12)−(11) FT	(14) $H^{1/2}$	(15) Q CFS
67 20	0.676	0.107	3 96	0.100	3.70	−0.0065	−0.24	291.70	287.76	289.73	300	10.27	3.20	2,900
											315	25.27	5.03	4,500
											325	35 27	5.94	5,400
76.06	0.683	0.205	7.59	0.200	7.40	−0.0035	−0.13	295.40	287.87	291.64	310	18.36	4.28	7,500
											315	23.36	4.83	8,400
											325	33.36	5.78	10,100
83.98	0.694	0.304	11.25	0.300	11.10	−0.0022	−0.08	299.10	287.92	293.51	310	16.49	4.06	10,700
											315	21.49	4.64	12,200
											325	31.49	5.61	14,800
91 20	0.707	0.403	14.91	0.400	14.80	−0.0018	−0.07	302.80	287.93	295.37	315	19.63	4.43	15,800
											320	24.63	4.96	17,600
											325	29.63	5.44	19,300

* FROM HYDRAULIC DESIGN CHART 311-2
** FROM HYDRAULIC DESIGN CHART 311-1

TAINTER GATES ON SPILLWAY CRESTS
SAMPLE DISCHARGE COMPUTATIONS
HYDRAULIC DESIGN CHART 311-5

WES 3-56

HYDRAULIC DESIGN CRITERIA

SHEETS 311-6 AND 311-6/1

CREST PRESSURES

1. General. Pressures on standard spillways with partly open tainter gates are principally affected by the gate opening, gate geometry, and head on the gate. The effects of gate radius and trunnion elevation can be generally neglected within the limits of practical design.

2. Background. A laboratory study of the effects of gate seat location on pressures for standard shaped spillway crests (HDC 111-1 to 111-2/1) was made at WES[1] prior to 1948. A design head of 0.75 ft was used. The results of an extensive study by Lemos[2] of all geometric variables including gate seat locations upstream and downstream of the crest were published in 1965. A design head of 0.5 ft was used in this study. Comparable model[3] and prototype[4] data are also available.

3. Design Criteria. Dimensionless crest pressure profiles for small, medium, and large gate openings for the design head and 1.33 times the design head are given in HDC 311-6 and 311-6/1. The data are for gate seat locations of from $0.0H_d$ to $0.6H_d$ downstream of the crest. The study by Lemos[2] included gate seat locations from $-0.2H_d$ upstream to $0.6H_d$ downstream of the crest, gate radii of 1.0 and $1.25H_d$, trunnion elevations of from 0.2 to $1.0H_d$ above the crest, and heads of 1.0 and 1.25 H_d. Lemos' results indicate that the minor relative differences in gate radii, trunnion elevations, and gate openings of the experimental data shown on charts 311-6 and 311-6/1 should have negligible effect on crest pressures estimated from the charts. The Chief Joseph[3] and Altus[4] model curves were interpolated from observed data.

4. Application. The data given in the charts should be adequate for estimating crest pressures to be expected under normal design and operating conditions. When unusual design or operating conditions are encountered, the extensive work of Lemos can be used as a guide in estimating pressure conditions to be expected.

5. The data presented in charts 311-6 and 311-6/1 show that crest pressures resulting from normal design and operation practices are not controlling design factors. For partial gate openings the expected minimum crest pressures may range from about $-0.1H_d$ for pools at design head to about $-0.2H_d$ for heads approximating $1.3H_d$. Gated spillways are presently being built with 50-ft design heads; so for an underdesigned crest, the minimum pressure to be expected with gate control would be about -10 ft of water. This pressure would increase to -5 ft if design head was the maximum operating head. Pressures of these magnitudes should be free of cavitation. Periodic surges upstream of partially open tainter gates have been observed for certain combinations of head and gate width. Criteria for

surge prevention are given in ETL 1110-2-51.[5]

6. The pressure profiles in charts 331-6 and 311-6/1 can be used to estimate crest pressures for the design head for various gate openings and gate seat locations. The general absence of excessive negative pressures is noteworthy. Structural economy should no doubt have a strong influence on the selection of the gate seat location.

7. References.

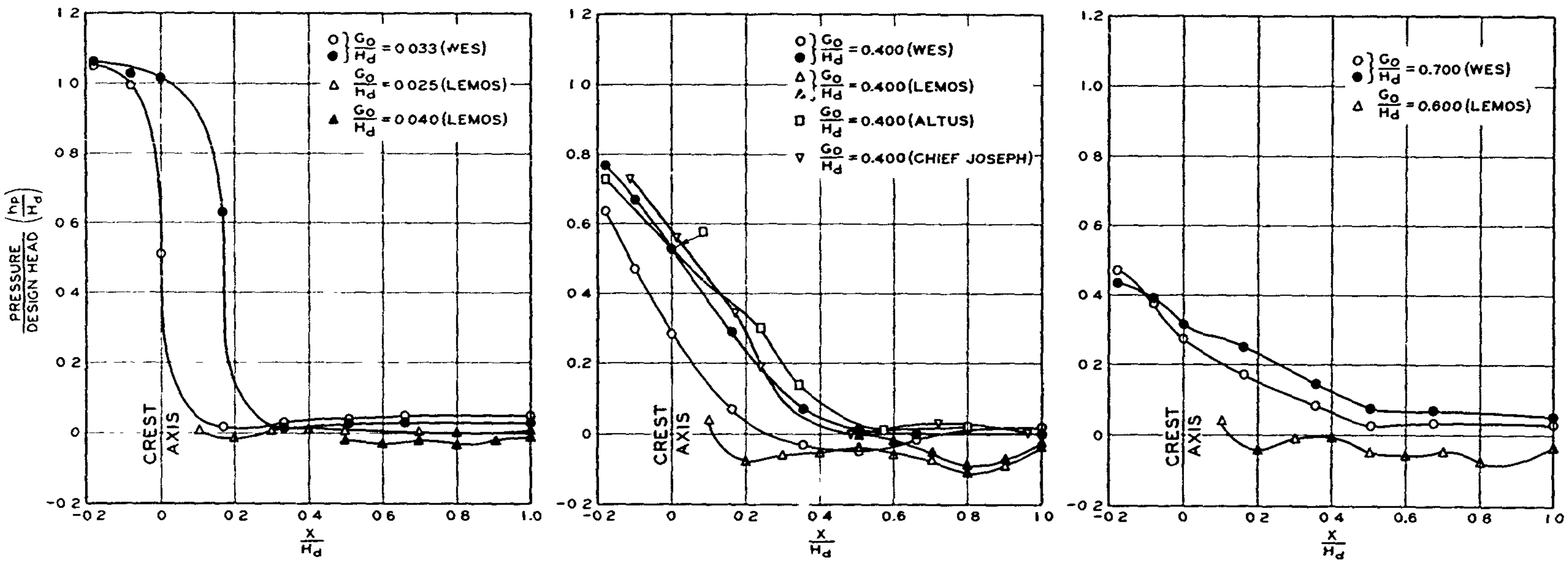
(1) U. S. Army Engineer Waterways Experiment Station, CE, General Spillway Tests (CW 801). Unpublished data.

(2) National Laboratory of Civil Engineering, Department of Hydraulics, Ministry of Public Works, Instability of the Boundary Layer - Its Effects Upon the Concept of Spillways of Dams, by F. O. Lemos. Proceedings 62/43, Lisbon, Portugal, 1965. WES Translation No. 71-3 by Jan C. Van Tienhoven, August 1971.

(3) U. S. Army Engineer Waterways Experiment Station, CE, Prototype Spillway Crest Pressures, Chief Joseph Dam, Columbia River, Washington. Miscellaneous Paper No. 2-266, Vicksburg, Miss., April 1958.

(4) Rhone, T. J., "Problems concerning use of low head radial gates." Proceedings of the American Society of Civil Engineers, Journal of the Hydraulics Division, paper 1935, vol 85, No. HY2 (February 1959).

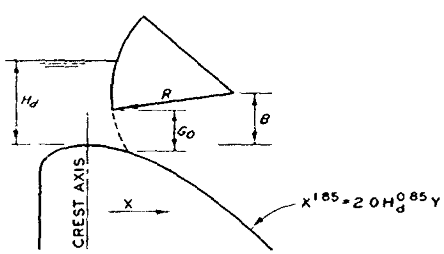
(5) U. S. Army, Office, Chief of Engineers, Engineering and Design; Design Criteria for Tainter Gate Controlled Spillways. Engineer Technical Letter No. 1110-2-51, Washington, D. C., 22 August 1968.



LEGEND

SYMBOL	TEST	GATE SEAT (X/H_d)	R/H_d	B/H_d
○———○	CW 801 (M)	0.000	1.27	0.385
●———●	CW 801 (M)	0.167	1.27	0.367
△———△	LEMOS (M)	0.000	1.25	0.560
▲———▲	LEMOS (M)	0.400	1.25	0.520
▽———▽	CHIEF JOSEPH (P) (INTERPOLATED)	0.258	1.00	0.444
□———□	ALTUS (M) (INTERPOLATED)	0.342	1.27	0.500

(M) MODEL
(P) PROTOTYPE



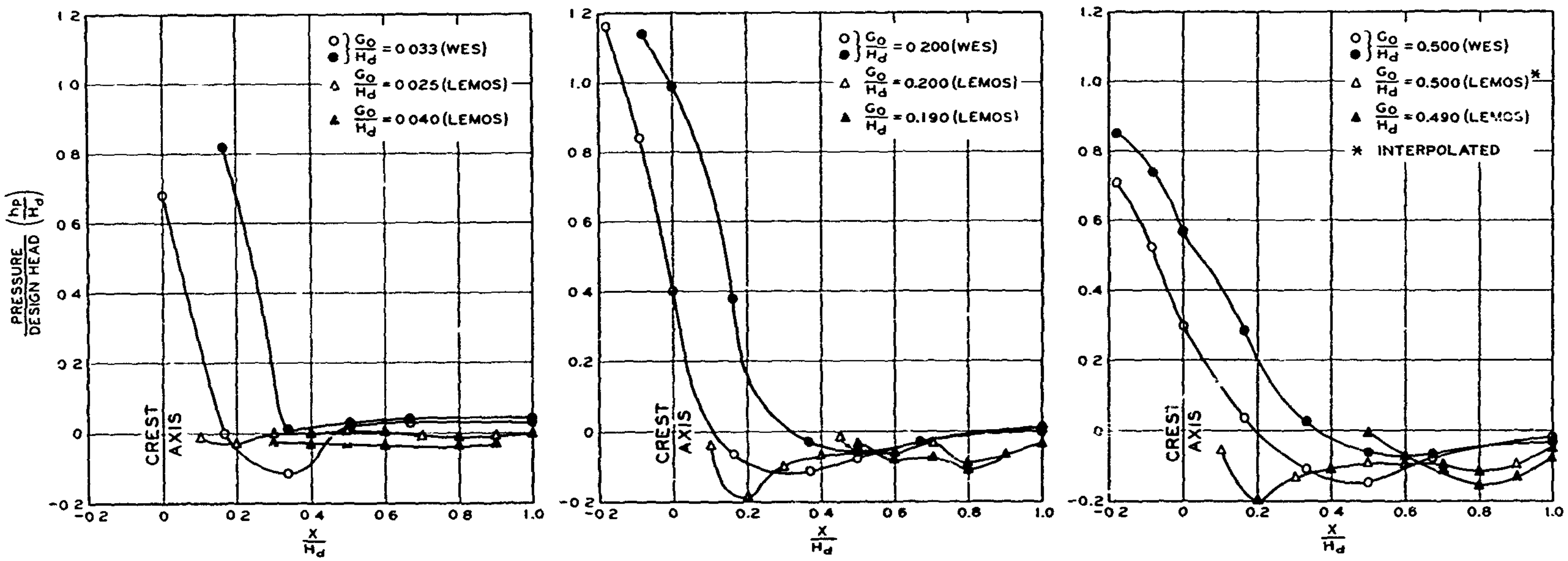
TAINTER GATES ON SPILLWAY CRESTS
EFFECT OF GATE SEAT LOCATION ON CREST PRESSURES FOR $H = 1.00 H_d$

HYDRAULIC DESIGN CHART 311-6

PREPARED BY U S ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

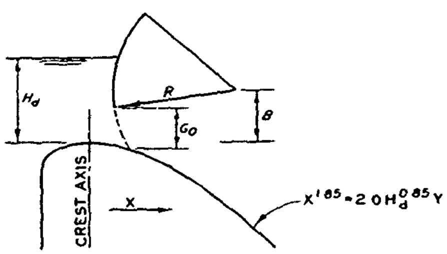
REV 7-71

WES 8-60



LEGEND

SYMBOL	TEST	GATE SEAT (X/H_d)	R/H_d	B/H_d	H/H_d
○—○	CW 801 (M)	0.000	1.27	0.385	1.33
●—●	CW 801 (M)	0.167	1.27	0.367	1.33
△—△	LEMOS (M)	0.000	1.25	0.560	1.25
▲—▲	LEMOS (M)	0.400	1.25	0.520	1.25



TAINTER GATES ON SPILLWAY CRESTS
EFFECT OF GATE SEAT LOCATION ON
CREST PRESSURES FOR $H \sim 1.3 H_d$

HYDRAULIC DESIGN CHART 311-6/1

PREPARED BY U S ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG, MISSISSIPPI

WES 7-71

HYDRAULIC DESIGN CRITERIA

SHEET 312

VERTICAL LIFT GATES ON SPILLWAYS

DISCHARGE COEFFICIENTS

1. Purpose. Vertical lift gates have been used on high-overflow-dam spillways. However, they are more commonly found on low-ogee-crest dams and navigation dams with low sills where reservoir pool control normally requires gate operation at partial openings. Hydraulic Design Chart 312 provides a method for computing discharge for partly opened, vertical lift gates.

2. Background. Discharge under high head, vertical lift gates can be computed using the standard orifice equation given in Sheets 311-1 to 311-5. The equation recommended by King[1] for discharge through low head orifices involves the head to the three-halves power. For flow under a low head gate, this equation can be expressed as

$$Q_G = C_{d1}\sqrt{2g}\, L \left(H_2^{3/2} - H_1^{3/2}\right) \qquad (1)$$

where Q_G is the gate controlled discharge, C_{d1} the discharge coefficient, g the acceleration of gravity, L the gate width, and H_1 and H_2 are the heads on the gate lip and gate seat, respectively.

3. A recent U. S. Army Engineers Waterways Experiment Station[2] study of discharge data from four laboratory investigations[3-6] failed to indicate correlation of discharge coefficients computed using equation 1 above or the equation given in Sheets 311-1 to 311-5. However, the concept of relating gate-controlled discharge to free discharge was developed in that study. The free discharge equation is

$$Q = C_d \sqrt{2g}\, LH^{3/2} \qquad (2)$$

where H is the head on the crest. The relation of controlled to free discharge was obtained by dividing equation 1 by equation 2.

$$\frac{Q_G}{Q} = \frac{C_{d1}}{C_d}\left(\frac{H_2^{3/2} - H_1^{3/2}}{H^{3/2}}\right) \qquad (3)$$

4. Analysis. The analysis of data taken from references 3 through 7 indicated reasonable correlation between free and controlled discharge. The results are shown in Chart 312. This study indicated that the relation C_{d1}/C_d varied slightly with the discharge ratio but could be assumed

as unity. Data from studies[6,7] with the gate seat located appreciably downstream from the crest showed good correlation with data for on-crest gate seat locations.

5. Application. Application of Chart 312 to the gate-discharge problem requires information on the head-discharge relation for free overflow for the crest under consideration. These data are usually available from spillway rating curves. Chart 312 should be a useful tool for the development of rating curves for vertical lift gates.

6. References.

(1) King, H. W., Handbook of Hydraulics for the Solution of Hydraulic Problems, revised by E. F. Brater, 4th ed. McGraw-Hill Book Co., Inc., New York, N. Y., 1954, pp 3-9.

(2) U. S. Army Engineer Waterways Experiment Station, CE, Discharge Rating Curves for Vertical Lift Gates on Spillway Crests, by R. H. Multer. Miscellaneous Paper No. 2-606, Vicksburg, Miss., October 1963.

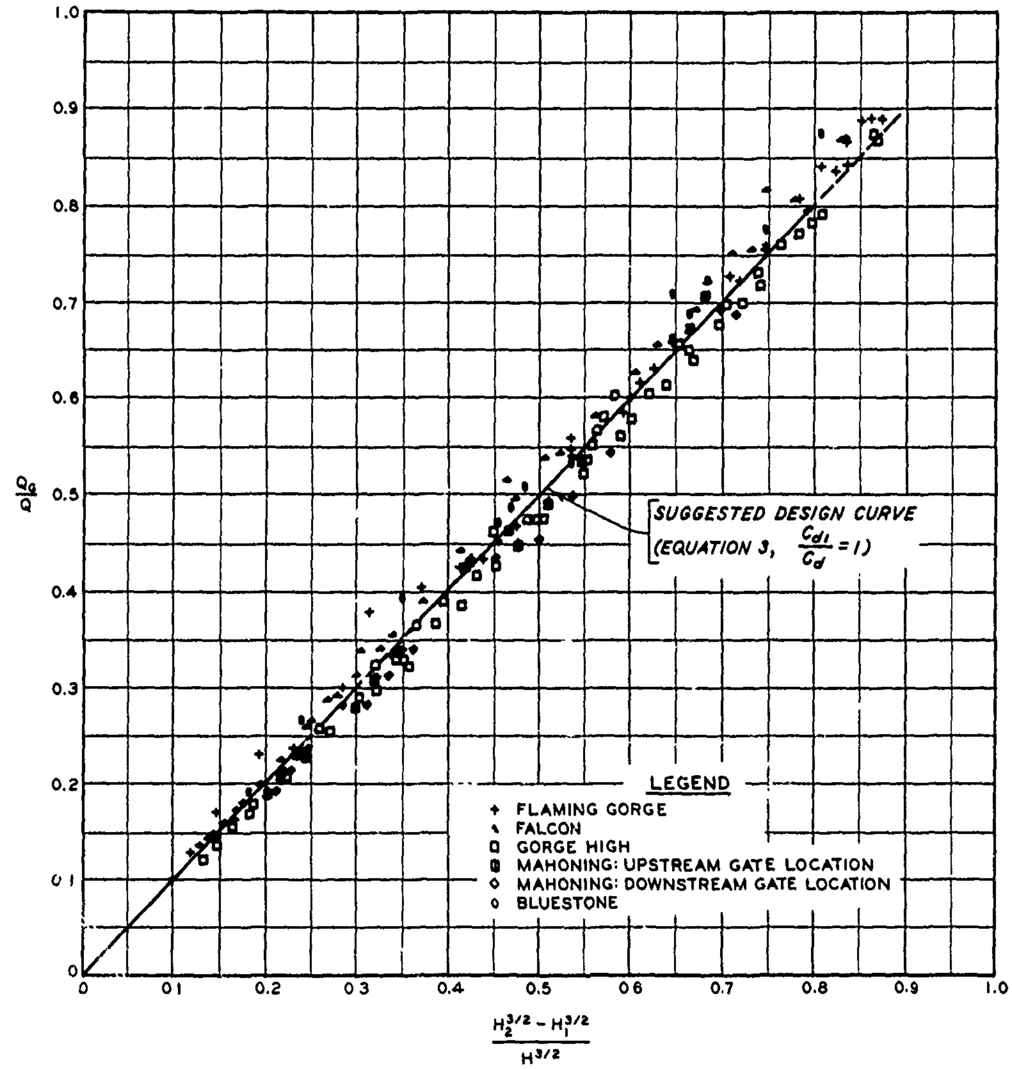
(3) U. S. Bureau of Reclamation, Hydraulic Model Studies of Falcon Dam, by A. S. Reinhart. Hydraulic Laboratory Report No. HYD-276, July 1950.

(4) _________, Hydraulic Model Studies of Gorge High Dam Spillway and Outlet Works, by W. E. Wagner. Hydraulic Laboratory Report No. HYD-403, September 1955.

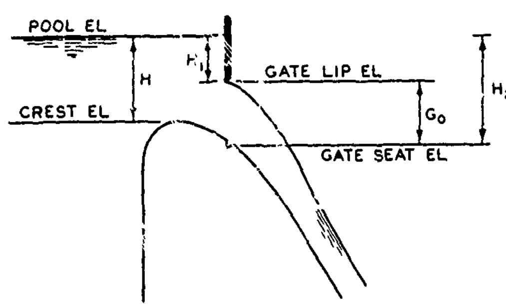
(5) Carnegie Institute of Technology, Laboratory Tests on Hydraulic Models of Bluestone Dam, New River, Hinton, W. Va. Final report, prepared for the U. S. Army Engineer District, Huntington, W. Va., February 1937.

(6) Case School of Applied Science, A Report on Hydraulic Model Studies for the Spillway and Outlet Works of Mahoning Dam on Mahoning Creek, Near Punxsutawney, Pa., by G. E. Barnes. Prepared for the U. S. Army Engineer District, Pittsburgh, Pa., May 1938.

(7) U. S. Bureau of Reclamation, Hydraulic Model Studies of Flaming Gorge Dam Spillway and Outlet Works, by T. J. Rhone. Hydraulic Laboratory Report No. HYD-531, May 1964.



NOTE: Q = FREE-FLOW DISCHARGE AT HEAD H
Q_G = DISCHARGE AT HEAD H AND GATE OPENING G_O
$H_1 = H_2 - G_O$



DEFINITION SKETCH

VERTICAL LIFT GATES ON SPILLWAYS

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 312

REV 1-68 WES 1-68

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

HYDRAULIC DESIGN CRITERIA

SHEET 320-1

CONTROL GATES

DISCHARGE COEFFICIENTS

1. General. The accompanying Hydraulic Design Chart 320-1 represents test data on the discharge coefficients applicable to partial openings of both slide and tractor gates. The basic orifice equation is expressed as follows:

$$Q = C\, G_o\, B \sqrt{2gH'}$$

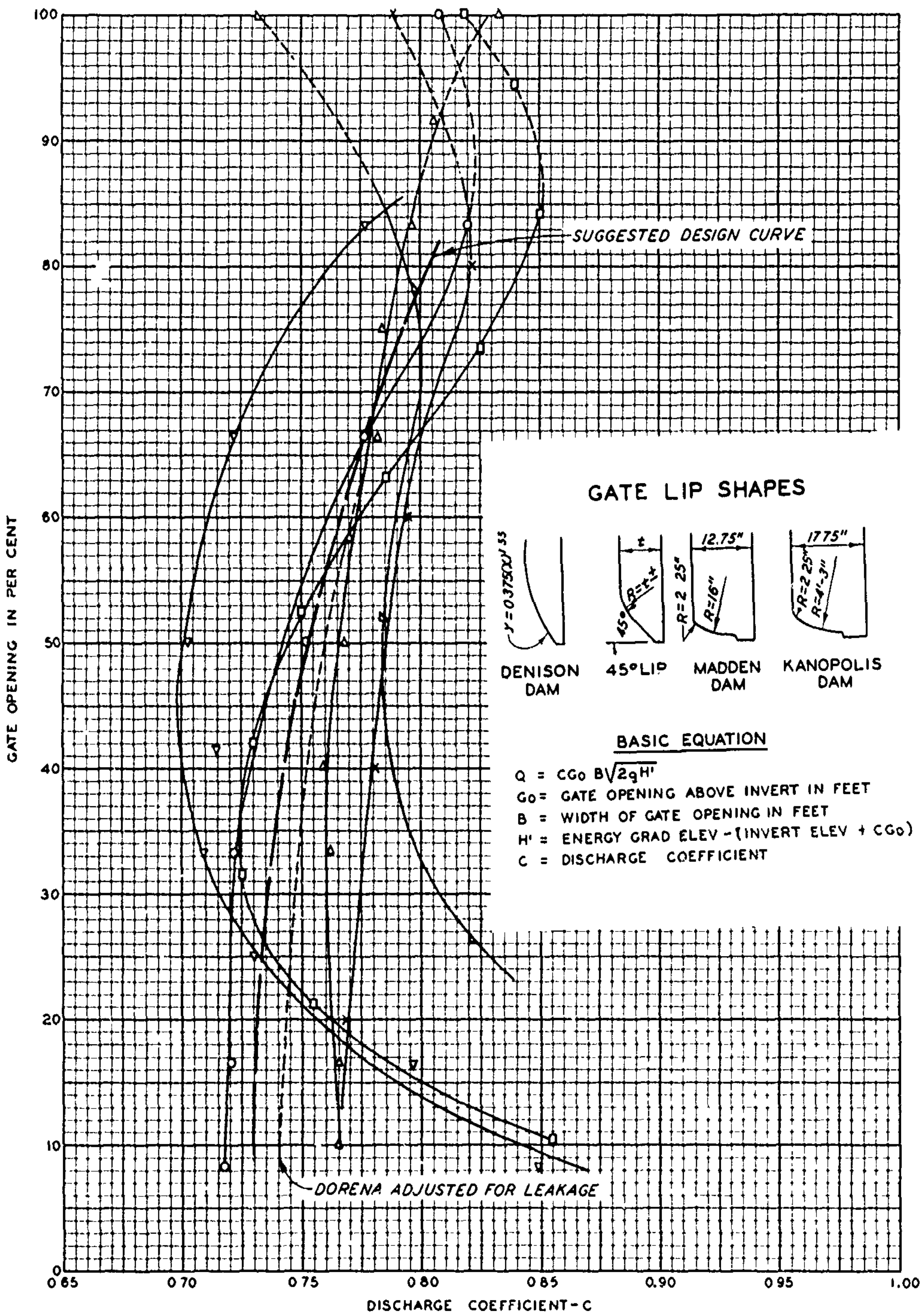
The coefficient C is actually a contraction coefficient if the gate is located near the tunnel entrance and the entrance energy loss is neglected. When the gate is located near the conduit entrance the head (H') is measured from the reservoir water surface to the top of the vena contracta. However, when the gate is located a considerable distance downstream of the conduit entrance, H' should be measured from the energy gradient just upstream of the gate to the top of the vena contracta because of appreciable losses upstream of the gate. The evaluation of H' requires successive approximation in the analysis of test data. However, the determination of H' in preparation of a rating curve can be easily accomplished by referring to the chart for C.

2. Discharge Coefficients. Discharge coefficients for tractor and slide gates are sensitive to the shape of the gate lip. Also, coefficients for small gate openings are materially affected by leakage over and around the gate. Chart 320-1 presents discharge coefficients determined from tests on model and prototype structures having various gate clearances and lip shapes. The points plotted on the 100 per cent opening are not affected by the gate but rather by friction and other loss factors in the conduit. For this reason the curves are shown by dashed lines above 85 per cent gate opening.

3. Suggested Criteria. Model and prototype tests prove that the 45° gate lip is hydraulically superior to other gate lip shapes. Therefore, the 45° gate lip has been recommended for high head structures. In the 1949 model tests leakage over the gate was reduced to a minimum. Correction of the Dorena Dam data for leakage results in a discharge coefficient curve that is in close agreement with the 1949 curve. The average of these two curves shown on Chart 320-1 is the suggested design curve. For small gate openings special allowances should be made by the designer for any expected excessive intake friction losses and gate leakage.

4. Values from the suggested design curve are tabulated below for the convenience of the designer.

Gate Opening, Per Cent	Discharge Coefficient
10	0.73
20	0.73
30	0.74
40	0.74
50	0.75
60	0.77
70	0.78
80	0.80



LEGEND

- ▲ FORT RANDALL MODEL } 45° LIP
- O WES MODEL TESTS CW 803 } 45° LIP
- Δ DORENA PROTOTYPE } 45° LIP
- □ DENISON PROTOTYPE
- X MADDEN PROTOTYPE
- ∇ KANOPOLIS PROTOTYPE

CONTROL GATES

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 320-1

WES 4-1-53

HYDRAULIC DESIGN CRITERIA

SHEETS 320-2 TO 320-2/3

VERTICAL LIFT GATES

HYDRAULIC AND GRAVITY FORCES

1. Purpose. The purpose of HDC's 320-2 to 320-2/2, which apply to the hydraulic forces on vertical lift gates, is to make the results of investigations of such forces available in a convenient nondimensional form. These charts are equally applicable to tractor gates and slide gates.

2. Definition. HDC 320-2 is included to simplify the definition of the hydraulic forces involved. For purposes of discussing buoyancy, a gate may be assumed to be a rectangular parallelepiped with the vertical axis coincident with the direction of gravity. If the body is completely inclosed, the buoyant force in still water is equal to the difference between the total pressure on top (downthrust) and the total pressure on the bottom (upthrust). For such an inclosed vertical body, water pressure on the upstream face has no vertical component of pressure.

3. Some engineers use the expression, the "wet weight" of a gate. This is simply the dry weight in air minus the buoyant force. If the body is cellular or lacks an upstream skin plate, the wet weight differs from that of a completely inclosed body. The gate shown in HDC 320-2 is an inclosed body and is further considered to have no horizontal projections such as gate seals.

4. The unit pressure on top of the gate, or downthrust, is dependent on the head of water in the gate well or the pressure head in the bonnet. This head in turn depends on the relation of the pressure difference across the gap and the area of the upstream gap coupled to the pressure differences and area of the downstream gap. Actually, the flow across the top of the gate has a hydrodynamic effect; but, for the purpose of these charts, this effect is not considered important.

5. The hydrodynamic effect of water flowing past the bottom of the gate is substantial. A reduction of pressure on the bottom from the theoretical static head is generally called "downpull," which may be viewed either as a reduction in upthrust or a reduction in buoyancy. Downpull is dependent upon the geometry of the gate bottom. HDC's 320-2 to 320-2/3 are concerned principally with the 45-degree gate bottom, for which experimental data are presented.

6. Vertical Stability. The gate well can be sucked completely dry of water with certain combinations of upstream and downstream gap areas between the gate and the roof of the conduit. If the upthrust then exceeds the weight of the gate, the entire body of the gate will be thrust

vertically upward. The experimental data on upthrust are of value in checking the design for such a possibility. However, discharge coefficients for the upstream and downstream gaps must be assumed to determine whether a gate opening exists that could cause a practically dry well.

7. Upthrust. Dimensionless plots of unit upthrust on the sloping bottom of four 45-degree gate-bottom designs are shown in HDC 320-2/1. The data sources are listed in paragraph 11. The data include both model and prototype pressure measurements. The Fort Randall gate has a downstream skin plate and downstream seals, and the 45-degree sloping gate bottom has an upstream skin plate. The Pine Flat and Norfork gates have upstream skin plates and downstream seals.

8. The upthrust force was computed from observed pressure data on the sloping gate bottom. These data were plotted on the horizontal plane of projection of the gate bottom. Pressure contours in feet of water were drawn, integrated, and divided by the area of projection between the conduit walls to determine the upthrust per unit area of cross section. The plots of data indicate that the conduit width-average gate thickness ratio is a factor in the magnitude of upthrust per unit area. The average gate thickness includes the gate bottom seal.

9. Pressure per unit area on top of the gate can be determined from HDC 320-2/2. The Fort Randall Dam data shown in the chart are based on field and model measurements of gate-well water-surface elevations. The Pine Flat and Norfork Dam data result from field measurements of bonnet pressures at these structures. Details of clearances between the gates and the gate recesses are also shown. The area of the top of the gate to be used in computation of the downthrust should include the area of the gate within the gate slots, the area between the conduit walls and the area of the gate top seal.

10. Application. HDC 320-2/3 is a sample computation illustrating the use of HDC's 320-2/1 and 320-2/2 in the solution of a hydraulic and gravity force problem. In this computation the hydraulic force is based on the cross-sectional area of the gate between the conduit walls. In actual design, the effects of the top and bottom gate seals and the area of the gate within the gate slots should also be considered.

11. Data Sources.

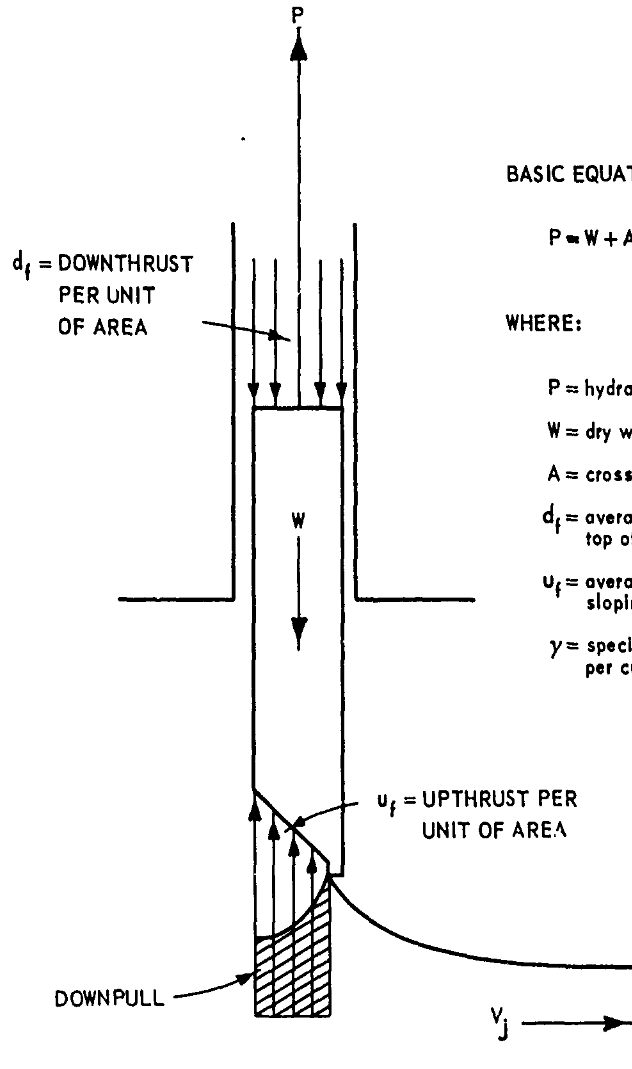
(1) U. S. Army Engineer Waterways Experiment Station, CE, Vibration, Pressure and Air-Demand Tests in Flood-Control Sluice, Pine Flat Dam, Kings River, California. Miscellaneous Paper No. 2-75, Vicksburg, Miss., February 1954, and subsequent unpublished test data.

(2) ________, Slide Gate Tests, Norfork Dam, North Fork River, Arkansas. Technical Memorandum No. 2-389, Vicksburg, Miss., July 1954.

(3) ________, Vibration and Pressure-Cell Tests, Flood-Control Intake

Gates, Fort Randall Dam, Missouri River, South Dakota. Technical Report No. 2-435, Vicksburg, Miss., June 1956.

(4) U. S. Army Engineer Waterways Experiment Station, CE, Spillway and Outlet Works, Fort Randall Dam, Missouri River, South Dakota. Technical Report No. 2-528, Vicksburg, Miss., October 1959.



BASIC EQUATION

$$P = W + A\,(d_f - u_f)\,\gamma$$

WHERE:

P = hydraulic and gravity forces in tons

W = dry weight of gate in tons

A = cross-sectional area of gate in sq ft

d_f = average downthrust per unit of area on top of gate in feet of water

u_f = average upthrust per unit of area on sloping bottom of gate in feet of water

γ = specific weight of water, 0.0312 ton per cu ft

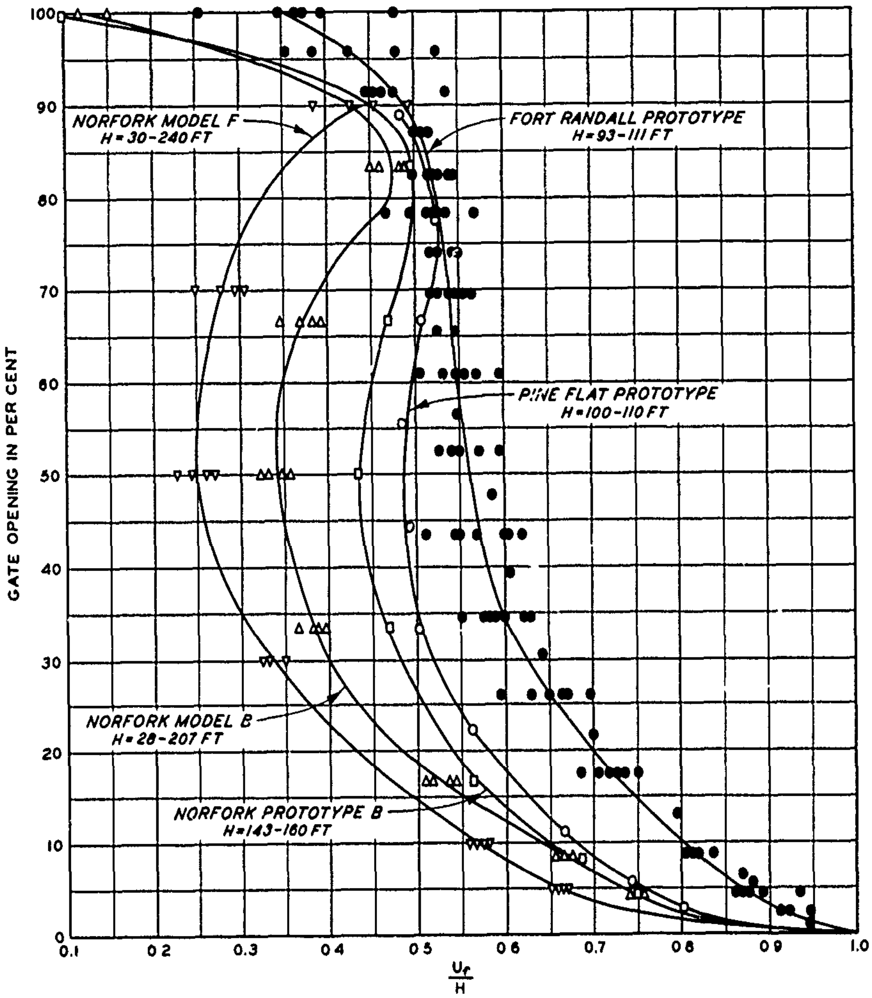
Note: Does not include factor for frictional and other mechanical forces.

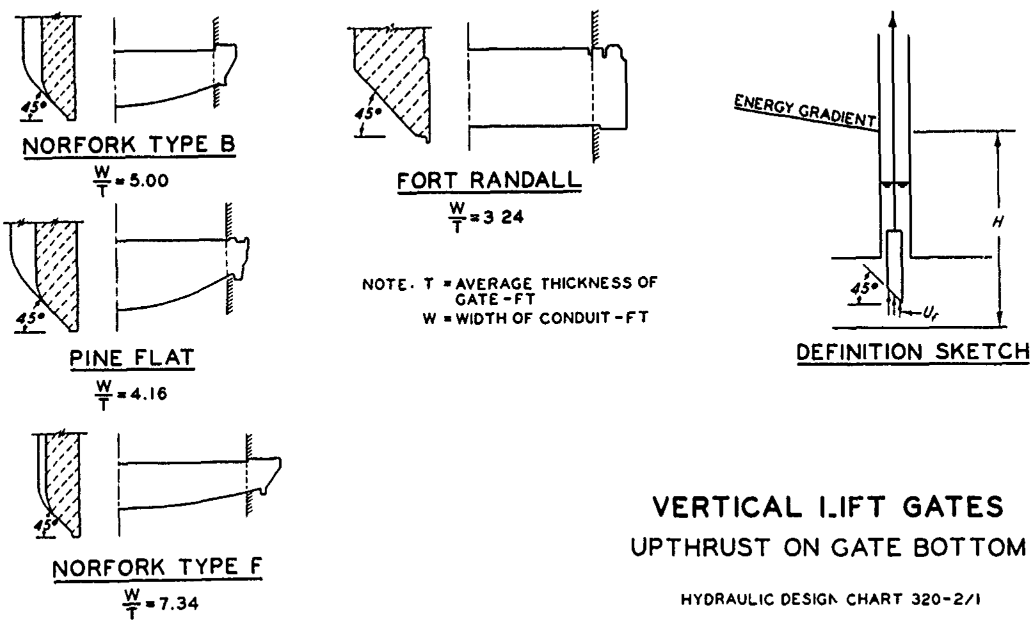
d_f = gate well water surface above conduit invert (H_w) minus sum of gate height (D) and gate opening (G_o).

VERTICAL LIFT GATES

HYDRAULIC AND GRAVITY FORCES DEFINITION AND APPLICATION

HYDRAULIC DESIGN CHART 320-2

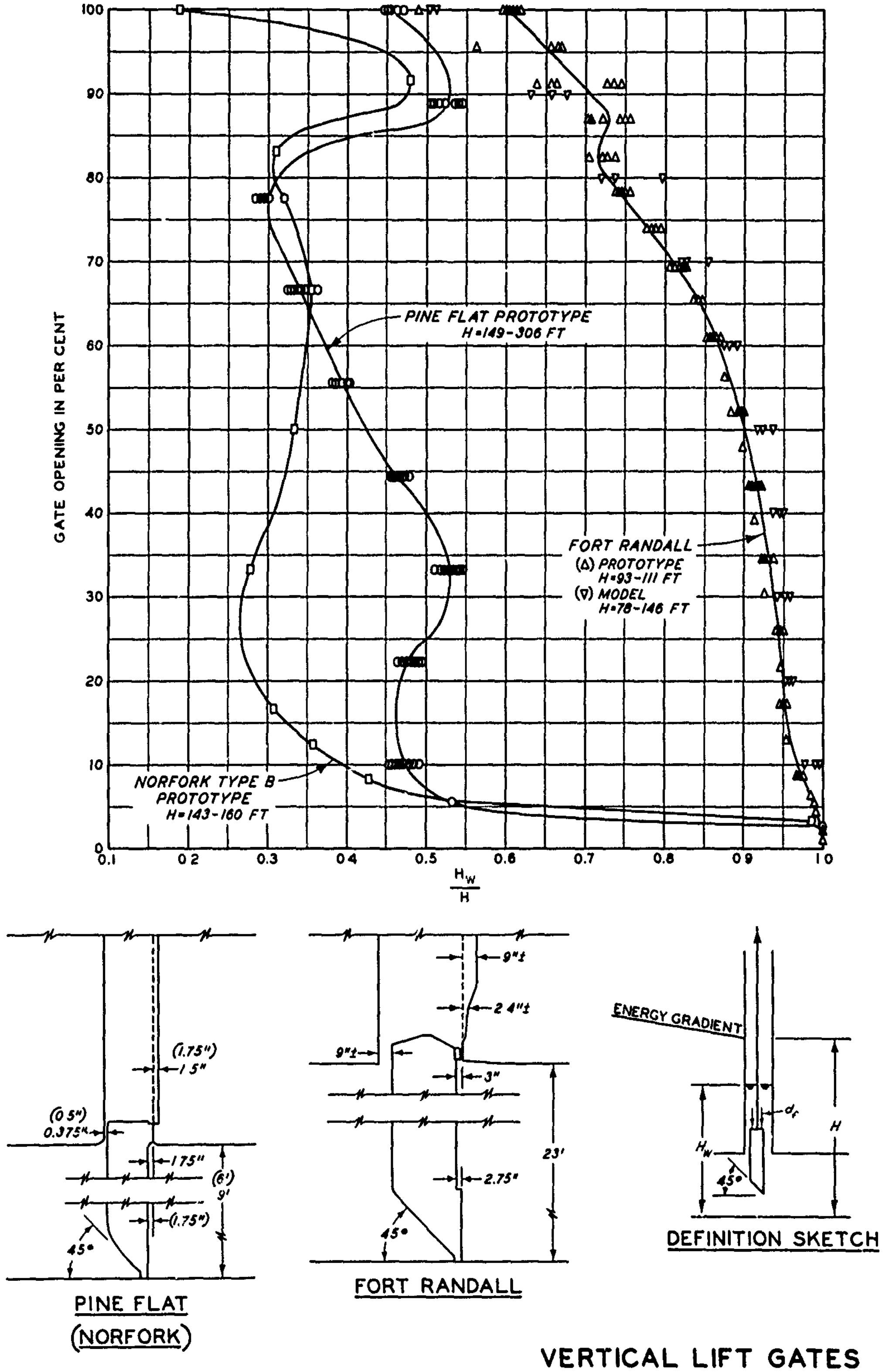




NOTE: T = AVERAGE THICKNESS OF GATE - FT
W = WIDTH OF CONDUIT - FT

VERTICAL LIFT GATES
UPTHRUST ON GATE BOTTOM

HYDRAULIC DESIGN CHART 320-2/1



VERTICAL LIFT GATES

GATE WELL WATER SURFACE

HYDRAULIC DESIGN CHART 320-2/2

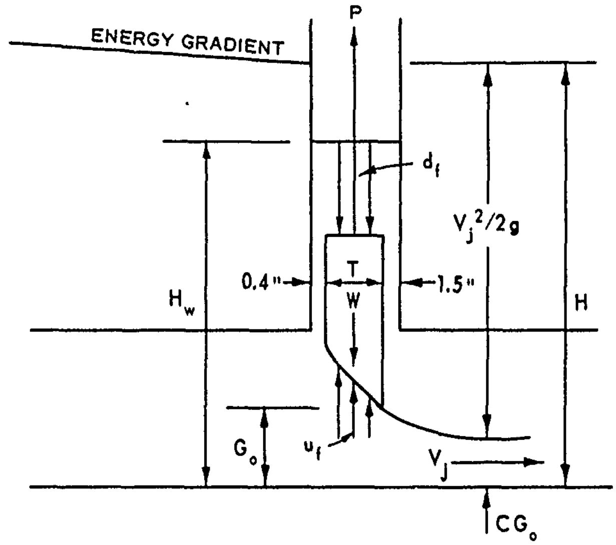
U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

JOB CW 804 PROJECT John Doe Dam SUBJECT Vertical Lift Gates
COMPUTATION Hydraulic and Gravity Forces
COMPUTED BY MBB DATE 4/10/61 CHECKED BY CWD DATE 4/20/61

GIVEN:

Gate - Pine Flat type (HDC 320-2/1)
- Height $(D) = 9.0$
- Width $(B) = 5.0$
- Average thickness $(T) = 1.2$ ft
- Upstream gate clearance = 0.4 in.
- Downstream gate clearance = 1.5 in.
- Dry weight $(W) = 8$ tons
- Gate opening $(G_o) = 3.0$ ft

Discharge $(Q) = 1200$ cfs



DETERMINE:

1. Energy head above conduit invert (H)

   Gate opening (G_o) percent

   $$\frac{G_o}{D} \times 100 = \frac{3}{9} \times 100 = 33.3$$

   Gate coefficient $(C) = 0.737$ (HDC 320-1)

   Velocity of jet (V_j)

   $$\frac{Q}{CG_oB} = \frac{1200}{0.737 \times 3 \times 5} = 108.5 \text{ ft/sec}$$

   Velocity head of jet $(V_j^2/2g)$

   $$\frac{V_j^2}{2g} = \frac{(108.5)^2}{64.4} = 182.8 \text{ ft}$$

   Energy head above conduit invert

   $$H = CG_o + V_j^2/2g$$
   $$= (0.737 \times 3) + (182.8) = 185.0 \text{ ft}$$

2. Unit upthrust (u_f)

   For Pine Flat from HDC 320-2/1

   $$\frac{u_f}{H} = 0.51 \text{ for } G_o = 33.3 \text{ percent}$$
   $$u_f = 0.51\,(185.0) = 94.4 \text{ ft}$$

3. Unit downthrust (d_f)

   For Pine Flat from HDC 320-2/2

   Gate well water surface above conduit invert (H_w)

   $$\frac{H_w}{H} = 0.53 \text{ for } G_o = 33.3 \text{ percent}$$
   $$H_w = 0.53\,(185.0) = 98.0 \text{ ft}$$

   Unit downthrust

   $$d_f = H_w - (D + G_o) = 98.0 - (9 + 3)$$
   $$= 86.0 \text{ ft}$$

4. Hoist load (P) (HDC 320-2)

   $$P = W + A\,(d_f - u_f)\,\gamma$$
   $$= 8 + (5 \times 1.2)\,(86.0 - 94.4)\,0.0312$$
   $$= 8 - 1.6 = 6.4 \text{ tons}$$

5. Repeat computations for other gate openings to develop gate hoist load curve.

Note:
1. The vertical load resulting from the friction between the gate and the gate guides has not been included in this computation.
2. In actual problems the difference between the projected areas of the top and bottom of the gate including seals and areas within the gate slots should be considered.

VERTICAL LIFT GATES
HYDRAULIC AND GRAVITY FORCES
SAMPLE COMPUTATION

HYDRAULIC DESIGN CHART 320-2/3

HYDRAULIC DESIGN CRITERIA

SHEET 320-3

TAINTER GATES IN CONDUITS

DISCHARGE COEFFICIENTS

1. HDC 320-3 presents coefficient curves for tainter gates in conduits for use in the discharge equation:

$$Q = C\ G_o\ B\sqrt{2gH}$$

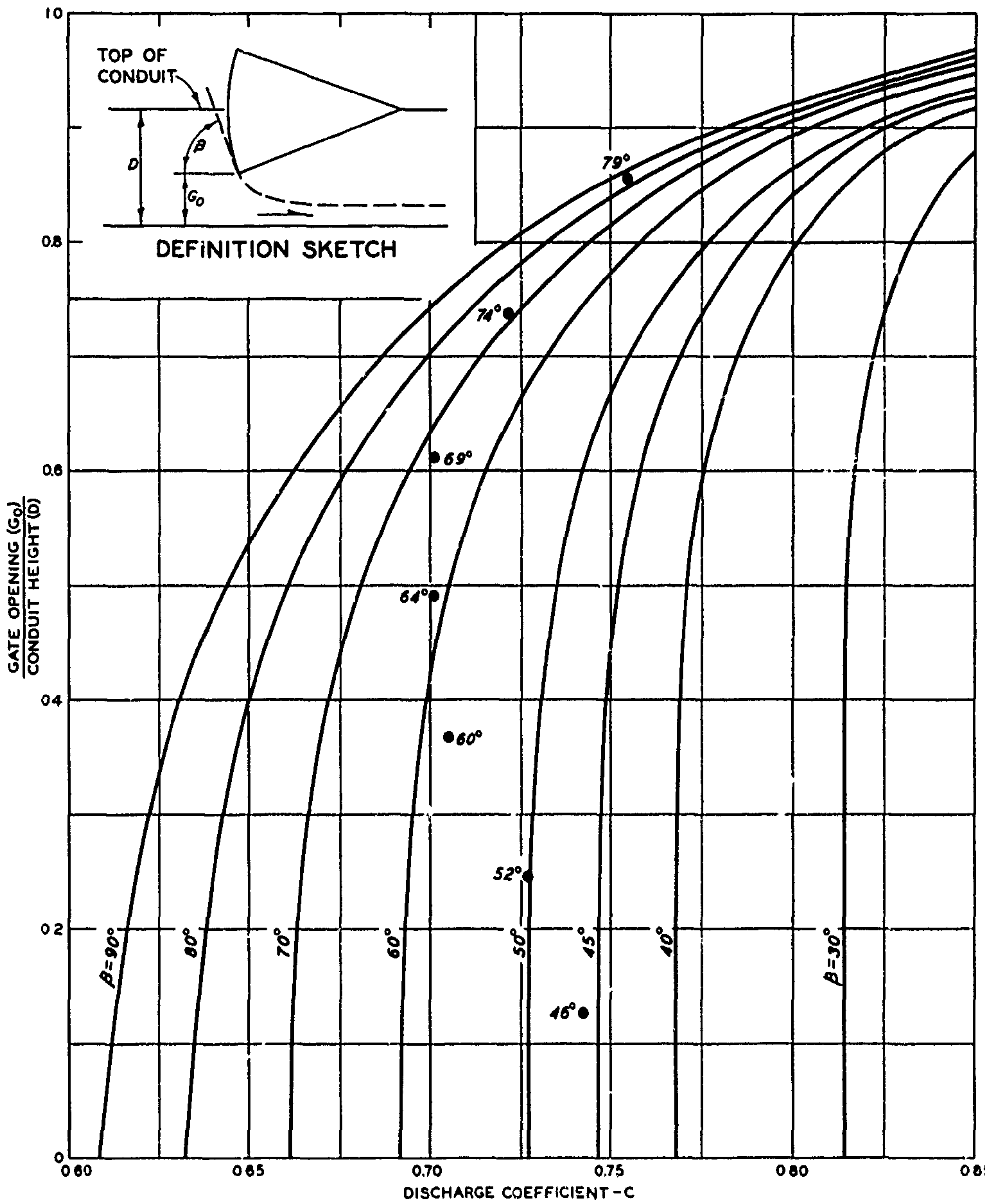
The coefficient C is actually a contraction coefficient when the head H is measured from the energy gradient just upstream from the gate to the top of the vena contracta downstream.

2. The curves shown in HDC 320-3 are based on an equation by R. von Mises* for the contraction coefficient for two-dimensional flow through slots. The solution of this equation requires successive approximation of the contraction coefficient. The computations were made on an electronic digital computer. The sketch shown in the chart is considered to be a half-section of the symmetrical slot condition investigated by Von Mises. The conduit invert represents the center line of his geometry and the roof one of the parallel approach boundaries. The tangent to the gate lip is assumed to be the sloping boundary from which the jet issues. The plotted data result from controlled tests on the Garrison tunnel model** in which leakage around or over the gate was negligible and discharge under the gate was carefully measured. The agreement between the curves and Garrison data indicates the applicability of the curves to tainter gates in conduits with straight inverts.

---

* Mises, R. von, "Berechnung von Ausfluss - und ueberfallzahlen (Computation of coefficients of out-flow and overfall)," Zeitschrift des Vereines deutscher Ingenieure, Band 61, Nr. 22 (2 June 1917), p 473.

** U. S. Army Engineer Waterways Experiment Station, CE, Outlet Works and Spillway for Garrison Dam, Missouri River, North Dakota, Technical Memorandum No. 2-431 (Vicksburg, Miss., March 1956).



BASIC EQUATION

$$Q = C G_0 B\sqrt{2gH}$$

WHERE .

- Q = DISCHARGE-CFS
- C = DISCHARGE COEFFICIENT
- G_0 = GATE OPENING-FT
- B = WIDTH OF GATE OPENING-FT.
- H = ENERGY GRAD. ELEV -(INVERT ELEV + CG_0)

LEGEND

- ——— VON MISES
- ● GARRISON MODEL

TAINTER GATES IN CONDUITS

DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 320-3

HYDRAULIC DESIGN CRITERIA

SHEETS 320-4 TO 320-7

TAINTER GATES IN OPEN CHANNELS

DISCHARGE COEFFICIENTS

1. Free discharge through a partially open tainter gate in an open channel can be computed using the equation:

$$Q = C_1 C_2 G_o B \sqrt{2gh}$$

The coefficient (C_1) depends on the vena contracta, the shape of which is a function of the gate opening (G_o), gate radius (R), trunnion height (a), and upstream depth (h) for gate sills at streambed elevations. When the gate sill is above streambed elevation, the coefficient also depends upon sill height (P) and sill length (L).

2. Hydraulic Design Charts 320-4 to 320-6 present discharge coefficients (C_1) for tainter gates with sills at streambed elevation. The insert graphs on the charts indicate adjustment factors (C_2) for raised sill conditions. Charts are included for a/R ratios of 0.1, 0.5, and 0.9. Coefficients for other a/R values can be obtained by interpolation between the charts. The coefficient is plotted in terms of the h/R ratio for G_o/R values of 0.05 to 0.5. The effect of G_o/h is inherent in the solution and is indicated by the limit-use curve $G_o/h = 0.8$.

3. The basic curves on Charts 320-4 to 320-6 were prepared from tests reported by Toch (3), Metzler (2), and Gentilini (1). The method of plotting was developed by Toch. Cross plots of the Toch, Metzler, and Gentilini data resulted in the interpolated curves. Good correlation of test results was obtained for the larger gate openings. Similar correlation was not obtained in all cases for the smaller gate openings. The Gentilini data for the smaller G_o/R ratios and their general correlation with Metzler's data resulted in the interpolated curves for G_o/R values of 0.05 and 0.1. The 0.2 curve is in close agreement with results reported by Toch. Interpolated coefficients from the C_1 curve indicate general agreement with experimental results to within ± 3 per cent.

4. Charts 320-4 to 320-6 also apply to raised sill design problems when the adjustment factor curve shown on the auxiliary graph is considered. The C_2 curve was developed from U. S. Army Corps of Engineers (4-7) studies and indicates the effects of the L/P ratio on the discharge coefficient. This adjustment results in reasonable agreement with experimental data. Sufficient information is not available to determine the effects, if any, of the parameter P/R.

5. Hydraulic Design Chart 320-7 is a sample computation sheet illustrating application of Charts 320-4 to 320-6.

6. References.

(1) Gentilini, B., "Flow under inclined or radial sluice gates - technical and experimental results." La Houille Blanche, vol 2 (1947), p 145. WES Translation No. 51-9 by Jan C. Van Tienhoven, November 1951.

(2) Metzler, D. E., A Model Study of Tainter Gate Operation. State University of Iowa Master's Thesis, August 1948.

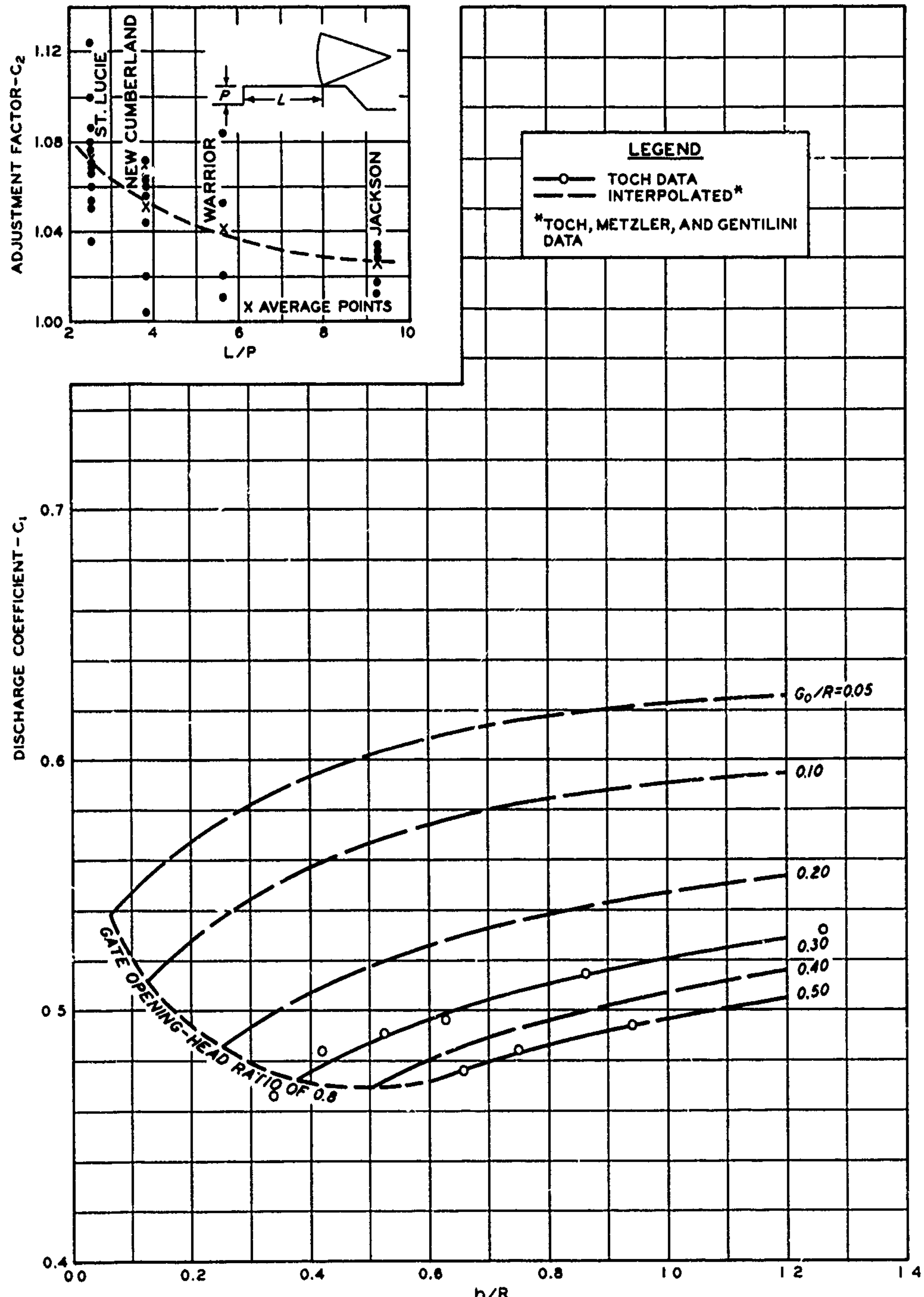
(3) Toch, A., The Effect of a Lip Angle Upon Flow Under a Tainter Gate. State University of Iowa Master's Thesis, February 1952.

(4) U. S. Army Engineer Waterways Experiment Station, CE, Model Study of the Spillway for New Lock and Dam No. 1, St. Lucie Canal, Florida. Technical Memorandum No. 153-1, Vicksburg, Miss., June 1939.

(5) __________, Spillway for New Cumberland Dam, Ohio River, West Virginia. Technical Memorandum No. 2-386, Vicksburg, Miss., July 1954.

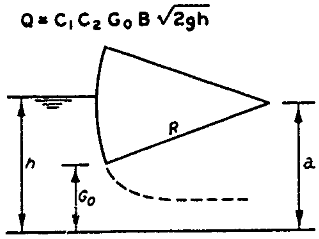
(6) __________, Stilling Basin for Warrior Dam, Warrior River, Alabama. Technical Report No. 2-485, Vicksburg, Miss., July 1958.

(7) __________, Spillways and Stilling Basins, Jackson Dam, Tombigbee River, Alabama. Technical Report No. 2-531, Vicksburg, Miss., January 1960.



BASIC EQUATION

$$Q = C_1 C_2 G_0 B \sqrt{2gh}$$



DEFINITION SKETCH

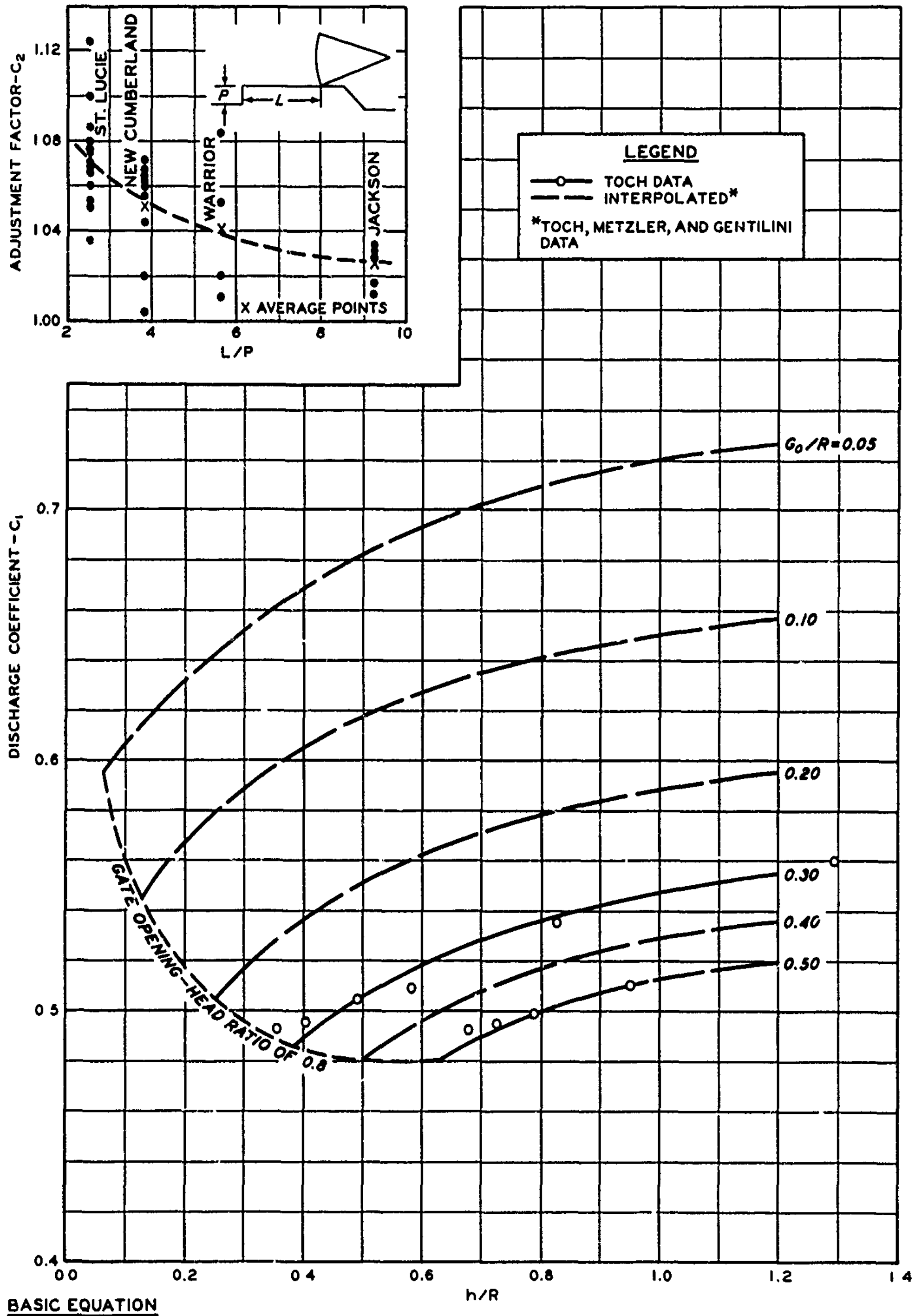
TAINTER GATE IN OPEN CHANNELS

DISCHARGE COEFFICIENTS

FREE FLOW

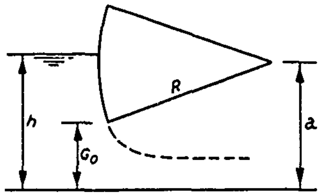
$a/R = 0.1$

HYDRAULIC DESIGN CHART 320-4



BASIC EQUATION

$$Q = C_1 C_2 G_0 B \sqrt{2gh}$$



DEFINITION SKETCH

TAINTER GATE IN OPEN CHANNELS

DISCHARGE COEFFICIENTS

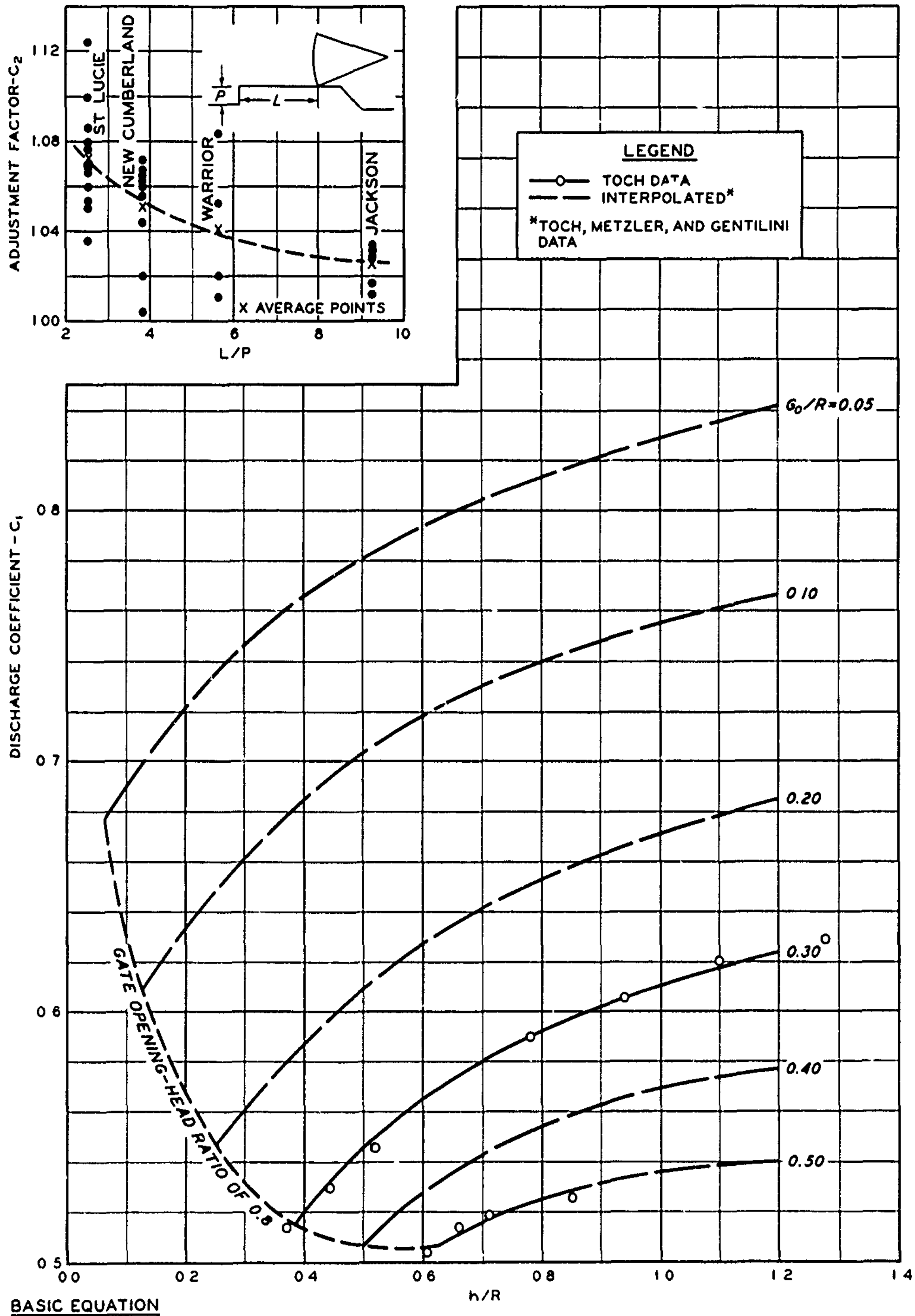
FREE FLOW

$a/R = 0.5$

HYDRAULIC DESIGN CHART 320-5

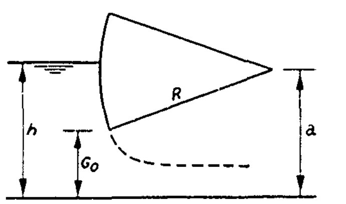
PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG, MISSISSIPPI

WES 8-60



BASIC EQUATION

$$Q = C_1 C_2 G_0 B \sqrt{2gh}$$



DEFINITION SKETCH

TAINTER GATE IN OPEN CHANNELS
DISCHARGE COEFFICIENTS
FREE FLOW
$a/R = 0.9$

HYDRAULIC DESIGN CHART 320-6

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG, MISSISSIPPI

WES 8-60

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

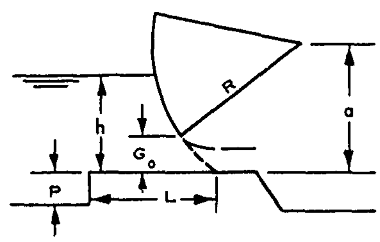
JOB CW 804 PROJECT John Doe River SUBJECT Tainter Gate in Open Channels

COMPUTATION Free Discharge for Gate Rating

COMPUTED BY MBB DATE 5/9/60 CHECKED BY RGC DATE 5/17/60

GIVEN:

Tainter gate installation as shown
Upstream depth $(h) = 15$ ft
Gate opening $(G_o) = 4$ ft
Gate radius $(R) = 25$ ft
Trunnion height $(a) = 20$ ft
Bay width $(B) = 60$ ft
Length – step to gate seat $(L) = 20$ ft
Height of step $(P) = 5$ ft



$Q = C_1 C_2 G_o B \sqrt{2gh}$

REQUIRED:

Free discharge for gate rating

COMPUTE:

1. Parameters

   $a/R = 0.8,\ h/R = 0.6,\ G_o/R = 0.16,\ L/P = 4$

2. Discharge coefficient (C_1) for unstepped condition for $a/R = 0.8$

   Chart 320-5 $(a/R = 0.5,\ h/R = 0.6,\ G_o/R = 0.16)$, $C_1 = 0.587$

   Chart 320-6 $(a/R = 0.9,\ h/R = 0.6,\ G_o/R = 0.16)$, $C_1 = 0.664$

   By interpolation for $a/R = 0.8$

   $$C_1 = 0.587 + \frac{0.8 - 0.5}{0.9 - 0.5}(0.664 - 0.587)$$
   $$= 0.645$$

3. Adjustment for stepped sill

   For $L/P = 4$

   Adjustment factor $(C_2) = 1.05$ (see chart insert)

   $C_1 C_2 = 0.645\ (1.05) = 0.678$

4. Discharge

   $$Q = C_1 C_2 G_o B \sqrt{2gh}$$
   $$= 0.678\ (4)\ (60) \sqrt{64.4 \times 15}$$
   $$= 5050 \text{ cfs}$$

TAINTER GATE IN OPEN CHANNELS
DISCHARGE COEFFICIENTS
FREE FLOW
SAMPLE COMPUTATION
HYDRAULIC DESIGN CHART 320-7

WES 8-60

HYDRAULIC DESIGN CRITERIA

SHEETS 320-8 AND 320-8/1

TAINTER GATES IN OPEN CHANNELS

DISCHARGE COEFFICIENTS

SUBMERGED FLOW

1. Tainter gates on low sills at navigation dams frequently operate at tailwater elevations resulting in submerged flow conditions. The discharge under the gate is controlled by the difference in the upper and lower pool elevations, the degree of sill submergence by the tailwater, the gate opening, and, to a lesser extent, the stilling basin apron elevation. Hydraulic Design Charts 320-8 and 320-8/1 present discharge coefficient data for computing flows under tainter gates on low sills operating under submerged conditions.

2. Basic Data. The U. S. Army Engineer Waterways Experiment Station (WES)[1] has developed the following equation for computing flows under gates on low sills with tailwater elevations greater than gate sill elevation.

$$Q = C_s L h_s \sqrt{2gh} \tag{1}$$

where

Q = discharge, cfs

C_s = submerged flow discharge coefficient, a function of the sill submergence-gate opening ratio

L = bay width, ft

h_s = tailwater depth over sill, ft

g = acceleration, gravitational, ft per sec^2

h = total head differential pool to tailwater, ft (including approach velocity head)

Equation 1 results in good correlation of experimental data when C_s is plotted as a function of the submergence-gate opening ratio (h_s/G_o). The equation was developed by modifying the standard orifice equation as follows

$$Q = CLG_o \sqrt{2gh} \tag{2}$$

or

$$Q\left(\frac{G_o}{h_s}\right) = CLG_o\left(\frac{G_o}{h_s}\right)\sqrt{2gh}$$

$$Q = C_s LG_o\left(\frac{h_s}{G_o}\right)\sqrt{2gh}$$

$$Q = C_s Lh_s\sqrt{2gh} \qquad (3)$$

where

$C_s = C(G_o/h_s)$

G_o = gate opening

3. Chart 320-8 presents the results of extensive model tests[2,3,4,5] and limited prototype data.[6] The plotted curves are based on careful measurements and are believed to be representative of the best available data. The model data and most of the prototype data were obtained with the gates adjacent to the test gate open the same amount as the test gate. The plotted curves indicate the effects of the relation of the elevation of the stilling basin apron to that of the gate sill. The portions of the curves having C_s values less than 0.1 are based on prototype gate openings of 1 ft or less and on model gate openings of about 0.05 ft. The experimental data are omitted from this chart in the interest of clarity. Chart 320-8/1 is included to illustrate the degree of data correlation resulting in the curves presented in Chart 320-8.

4. Application. The suggested design curve in Chart 320-8 should be useful for developing pool regulation curves for navigation dam spillways consisting of tainter gates on low sills. The curves presented generally represent sill elevations about 5 ft above streambed and stilling basin apron elevations 3.5 to 31 ft below sill elevation. The Hannibal and Cannelton spillway sills are located about 15 and 19 ft above streambed, respectively. The height of the sill above the approach bed does not seem to be an important factor in submerged flow controlled by gates. However, the coefficient data presented include all the geometric effects of each structure as well as the effects of adjacent gate operation. The curve most applicable to spillway design conditions should be used for developing discharge regulation curves.

5. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Typical Spillway Structure for Central and Southern Florida Water-Control Project; Hydraulic Model Investigation, by J. L. Grace, Jr. Technical Report No. 2-633, Vicksburg, Miss., September 1963.

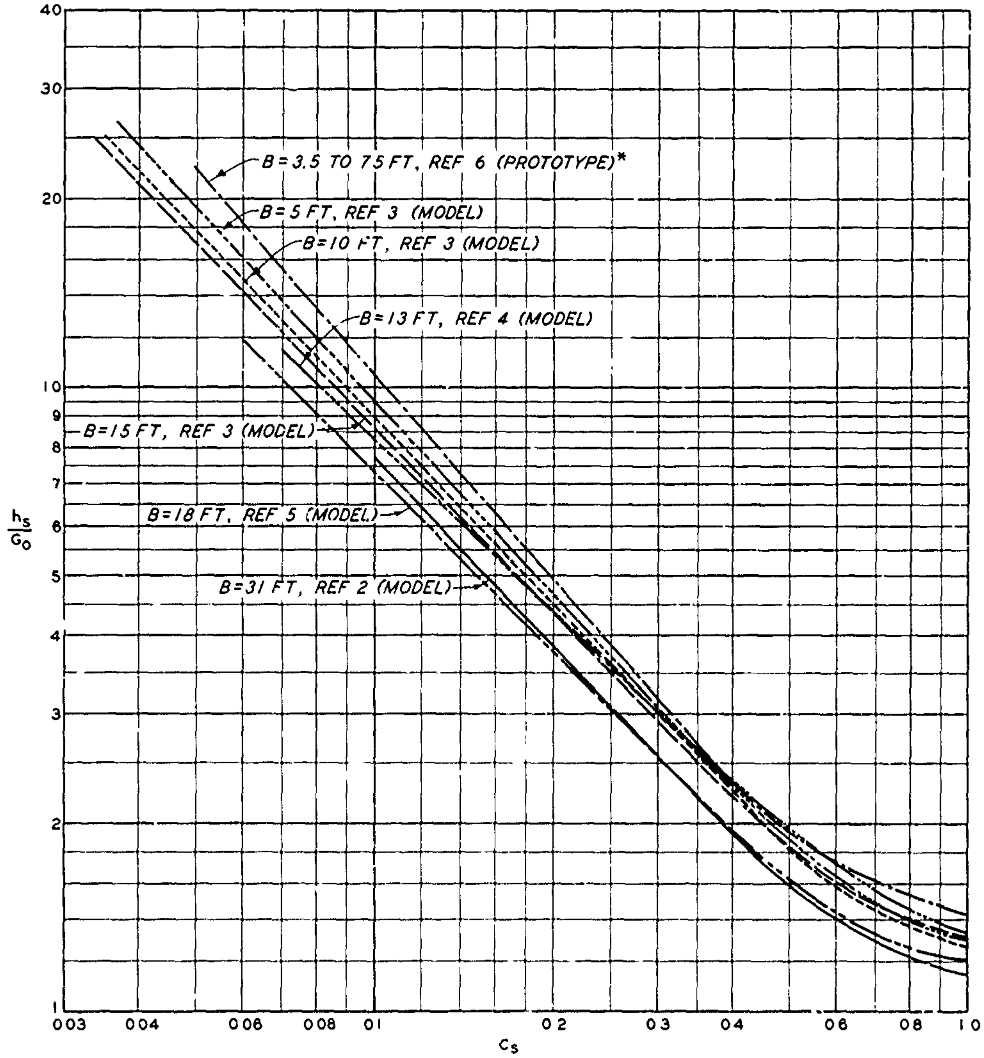
(2) __________, Spillway, Millers Ferry Lock and Dam, Alabama River, Alabama; Hydraulic Model Investigation, by G. A. Pickering. Technical Report No. 2-643, Vicksburg, Miss., February 1964.

(3) __________, Spillway for Typical Low-Head Navigation Dam, Arkansas River, Arkansas; Hydraulic Model Investigation, by J. L. Grace, Jr. Technical Report No. 2-655, Vicksburg, Miss., September 1964.

(4) __________, Spillway for Cannelton Locks and Dam, Ohio River, Kentucky and Indiana; Hydraulic Model Investigation, by G. A. Pickering and J. L. Grace, Jr. Technical Report No. 2-710, Vicksburg, Miss., December 1965.

(5) __________, Spillway, Hannibal Locks and Dam, Ohio River, Ohio and West Virginia; Hydraulic Model Investigation. Technical Report No. 2-731, Vicksburg, Miss., June 1966.

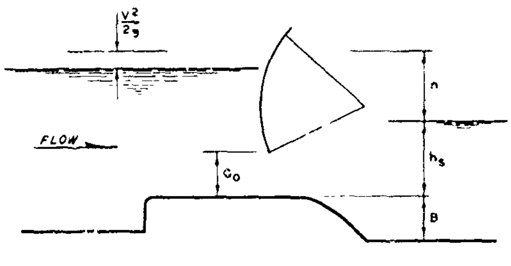
(6) Denzel, C. W., Submerged Tainter Gate Flow Calibration. 1965, U. S. Army Engineer District, St. Louis, Mo. (unpublished memorandum).



BASIC EQUATION

$$Q = C_s L h_s \sqrt{2gh}$$

*MISSISSIPPI RIVER DAMS 2, 5A, AND 26



DEFINITION SKETCH

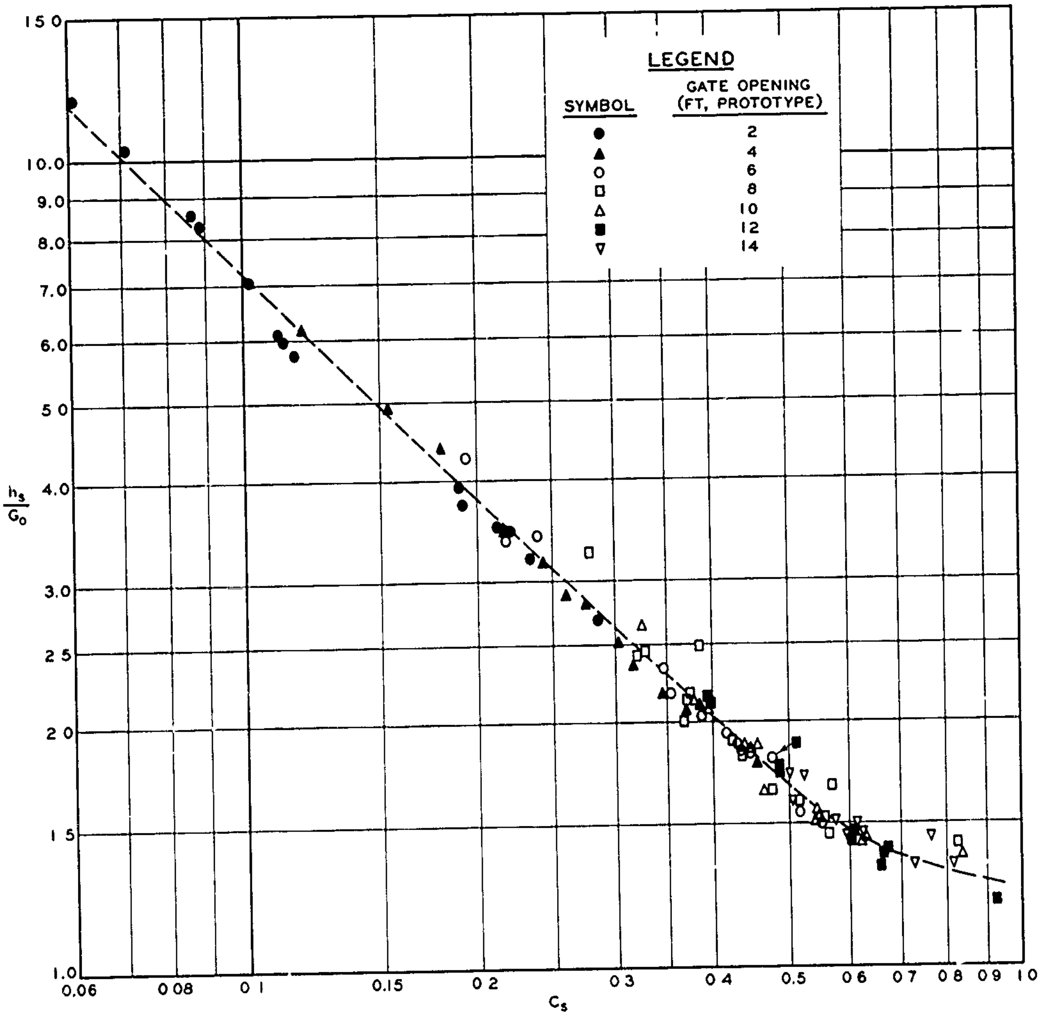
TAINTER GATES IN OPEN CHANNELS

DISCHARGE COEFFICIENT SUBMERGED FLOW

HYDRAULIC DESIGN CHART 320-8

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

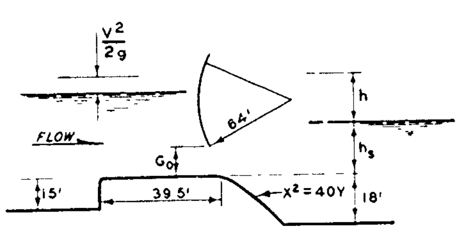
WES 1-68



BASIC EQUATION

$$Q = C_s L h_s \sqrt{2gh}$$

NOTE DATA FROM HANNIBAL MODEL, REF 5



DEFINITION SKETCH

TAINTER GATES IN OPEN CHANNELS

DISCHARGE COEFFICIENT SUBMERGED FLOW TYPICAL CORRELATION

HYDRAULIC DESIGN CHART 320-8/1

WES 1-68

PREPARED BY U S ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

HYDRAULIC DESIGN CRITERIA

SHEETS 330-1 AND 330-1/1

GATE VALVES

DISCHARGE CHARACTERISTICS

1. The discharge characteristics of a flow control valve may be expressed in terms of a loss coefficient for valves along a full-flowing pipeline, or in terms of a discharge coefficient for free flow from a valve located at the downstream end of a pipeline. Loss and discharge coefficients for gate valves are given on Hydraulic Design Charts 330-1 and 330-1/1, respectively.

2. Loss Coefficient. The loss of head caused by a valve occurs not only in the valve itself but also in the pipe as far downstream as the velocity distribution is distorted. Tests to determine this total loss, exclusive of friction, have been conducted on several makes and sizes of gate valves at the University of Wisconsin(1) and the Alden Hydraulic Laboratory.(2) The results of these tests on the larger sizes of valves are given on Chart 330-1 as loss coefficients in terms of the velocity head immediately upstream from the valve. Data are given for both a simple disk gate valve having a crescent-shaped water passage at partial openings and a ring-follower type of gate valve having a lens-shaped water passage at partial openings. The scatter in the Wisconsin data is attributed to minor variations in the geometry of the different makes of valves tested.

3. Discharge Coefficients. Discharge coefficients for free flow from a gate valve at the downstream end of a pipeline have been determined by the Bureau of Reclamation(3) for several makes and sizes of simple disk gate valves. The results of these tests are given on Chart 330-1/1 as discharge coefficients in terms of the total energy head immediately upstream from the valve. The scatter in these data is attributed to minor variations in geometry of the valves tested.

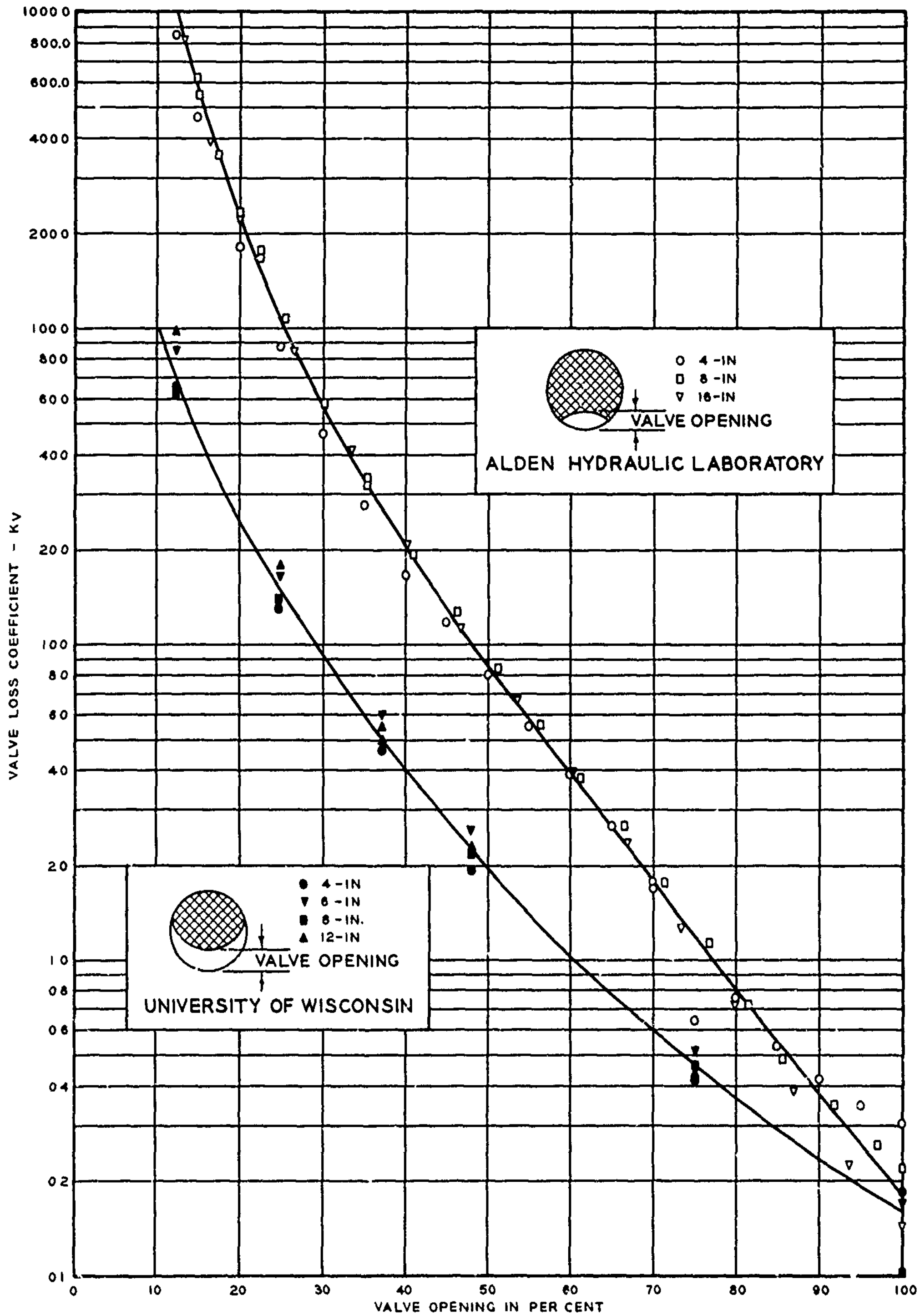
4. Application. The loss data given on Chart 330-1 are applicable to valves installed in full-flowing pipelines having no bends or other disturbances within several diameters upstream and downstream from the valve. The discharge coefficients on Chart 330-1/1 are for valves installed at the downstream end of several diameters of straight pipe and discharging into the atmosphere.

5. List of References.

(1) Corps, C. I., and Ruble, R. O., Experiments on Loss of Head in Valves and Pipes of One-half to Twelve Inches Diameter. University of Wisconsin Engineering Experiment Station Bulletin, vol. IX, No. 1, Madison, Wis., 1922.

(2) Hooper, L. J., Tests of 4-, 8-, and 16-Inch Series 600 Rising Stem Valves for the W-K-M Division of ACE Industries, Houston, Texas. Alden Hydraulic Laboratory, Worcester Polytechnic Institute, Worcester, Mass., Sept. 1949.

(3) U. S. Bureau of Reclamation, Study of Gate Valves and Globe Valves as Flow Regulators for Irrigation Distribution Systems Under Heads Up to About 125 Feet of Water. Hydraulic Laboratory Report No. Hyd-337, Denver, Colo., 13 Jan. 1956.



BASIC EQUATION $K_V = \frac{H_L}{V^2/2g}$

WHERE

K_V = VALVE LOSS COEFFICIENT

H_L - HEAD LOSS THROUGH VALVE

V = AVERAGE VELOCITY IN PIPE

NOTE

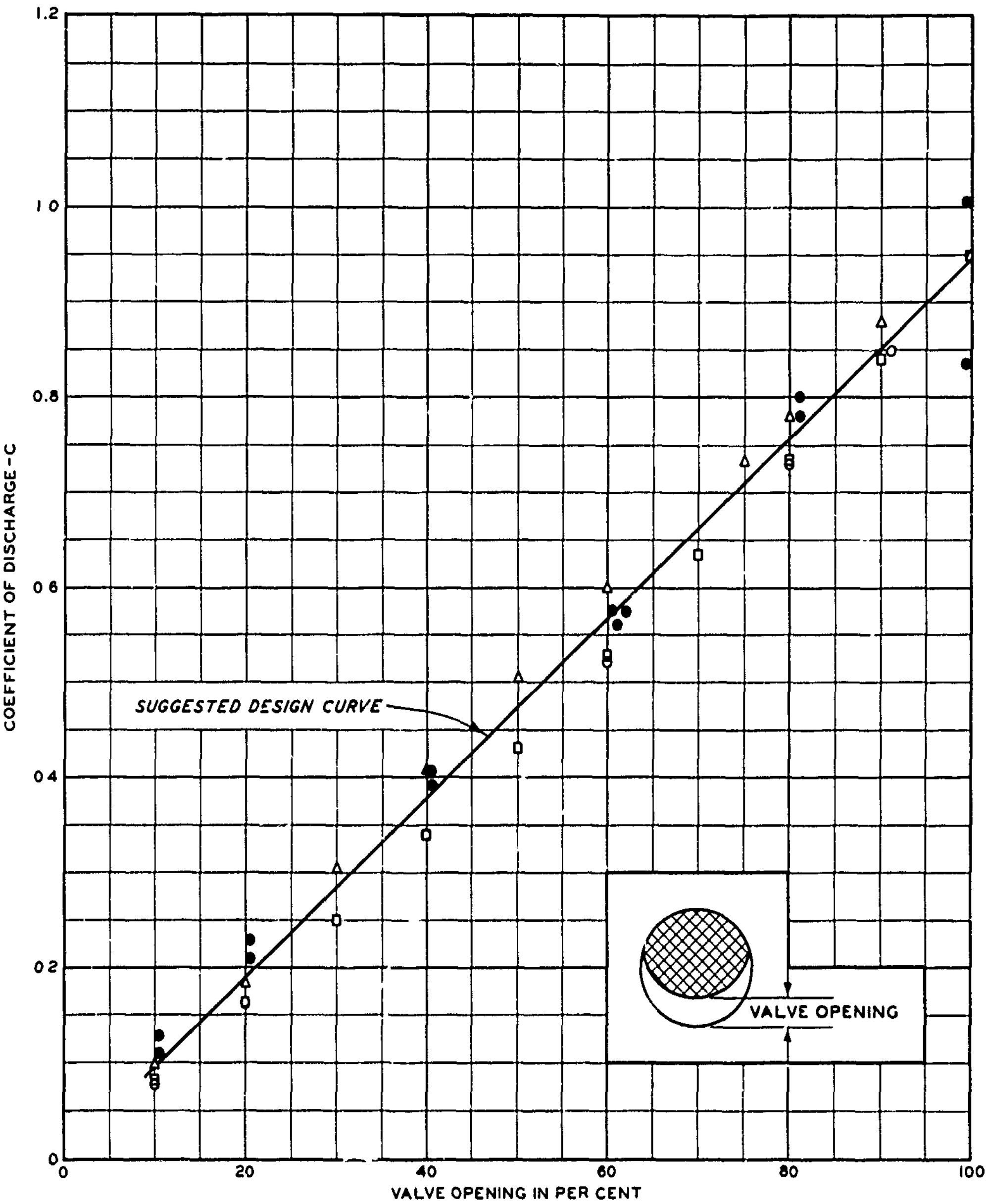
DATA ARE FOR VALVES HAVING SAME DIAMETER AS PIPE AND FOR DOWNSTREAM PIPE FLOWING FULL

GATE VALVES

LOSS COEFFICIENTS

HYDRAULIC DESIGN CHART 330-1

WES 6-57



BASIC EQUATION $Q = CA\sqrt{2gH_e}$

WHERE:

C = VALVE DISCHARGE COEFFICIENT

A = AREA BASED ON NOMINAL VALVE DIAMETER

H_e = ENERGY HEAD MEASURED TO CENTER LINE OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE

NOTE:

DATA ARE FROM USBR TESTS FOR FREE FLOW FROM 8- TO 12-INCH-DIAMETER GATE VALVES AT DOWNSTREAM END OF CONDUIT OF SAME NOMINAL DIAMETER AS VALVE

GATE VALVES
FREE FLOW
DISCHARGE COEFFICIENTS

HYDRAULIC DESIGN CHART 330-1/1

WES 6-57

HYDRAULIC DESIGN CRITERIA

SHEETS 331-1 to 331-3

BUTTERFLY VALVES

DISCHARGE AND HYDRAULIC TORQUE CHARACTERISTICS

1. The discharge and torque characteristics of butterfly valves can be expressed in terms of discharge and torque coefficients as functions of the angle of rotation of the valve vane from opened position. The discharge coefficient is primarily a function of the orifice opening whereas the hydraulic torque coefficient depends upon the geometry of the valve vane. Thus, differences in torque coefficients are to be expected for various shaped vanes at the same opening. Although considerable data have been published(2), only data indicated as the original computations or curves of the investigators have been included in Design Charts 331-1 to 331-2/1.

2. Discharge Coefficients. A modified form of the standard orifice equation has been used for computation of valve discharge. The area used in the equation is based on the nominal diameter of the valve because of difficulty in determining the actual areas of the orifice openings for partially opened valves. The discharge coefficient varies inversely with the angle of rotation of the valve from opened position. Two valve locations have been tested; one in which the valve is near the outflow end of the pipe, and the other in which the valve is well within a straight reach of pipe. Hydraulic Design Chart 331-1 presents discharge coefficients for valves located within the pipe. Chart 331-1/1 presents similar data for valves located near the end of the pipe. The material used in these charts is taken from the following investigators: McPherson(7), Dickey-Coplen(4), Gaden(5), Colleville(8), DeWitt(3), and Armanet(1). The Dickey-Coplen data are from air tests on a thin circular damper. The Armanet tests reflect the effects of convergence in the valve housing downstream from the vane pivot.

3. Torque Coefficients. Torque coefficient data are presented in Charts 331-2 and 2/1. The available information is limited. Chart 331-2 pertains to valves located within the pipe and Chart 331-2/1 applies to valves located near the end of the pipe. The Keller and Salzmann(6) data in Chart 331-2 were obtained from air tests. The DeWitt curve in Chart 331-2/1 was computed from published prototype torque curves. The Gaden curves are based on carefully controlled laboratory tests which included measurement of and correction for pressure distribution on the downstream face of the valve vane. The Armanet curves reflect the effects of convergence in the valve body. The scarcity of torque coefficient data is indicative of the need for torque tests on butterfly valves of American manufacture.

4. Application. A sample computation for torque is given in Chart 331-3. Final computations should be based on the recommendations of the valve manufacturer at which time friction torque and seating torque data should be considered.

5. List of References.

(1) Armanet, L., "Vannes-Papillon Des Turbines." Génissiat, Numéro Hors Série De La Houille Blanche, pp 199-219.

(2) Cohn, S. D., "Performance analysis of butterfly valves." Instruments, vol 24, No. 8 (August 1951), p 880-884.

(3) DeWitt, C., "Operating a 24-in. butterfly valve under a head of 223 ft." Engineering News-Record (18 September 1930), pp 460-462.

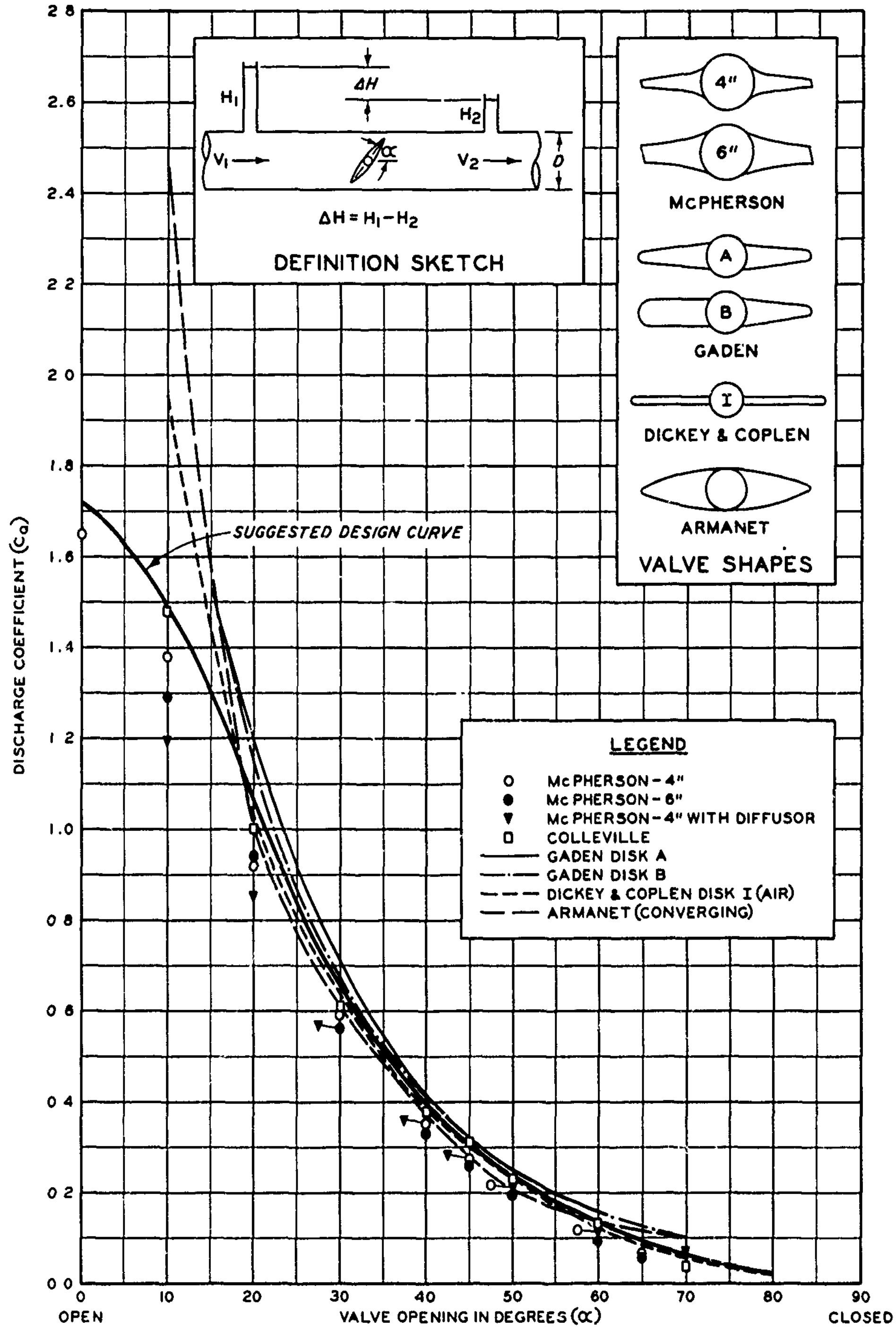
(4) Dickey, P. S., and Coplen, H. L., "A study of damper characteristics." Transactions, ASME, vol 64, No. 2 (February 1942).

(5) Gaden, D., "Contribution to study of butterfly valves." Schweizerische Bauzeitung, vol III, Nos. 21, 22, and 23 (May 21 and 28 and June 4, 1938). Similar material by D. Gaden was also published in England in Water Power (December 1951 and January 1952).

(6) Keller, C., and Salzmann, F., "Aerodynamic model tests on butterfly valves." Escher-Wyss News, vol IX, No. 1 (January-March 1936).

(7) McPherson, M. B., Strausser, H. S., and Williams, J. C., Jr., "Butterfly valve flow characteristics." Proceedings, ASCE, paper 1167, vol 83, No. HY1 (February 1957).

(8) Voltmann, Henry, discussion of reference 7. Proceedings, ASCE, vol 83, No. HY4 (August 1957), pp 1348-48 and 49.



BASIC EQUATION

$$Q = C_Q D^2 \sqrt{g} \sqrt{\Delta H}$$

WHERE.

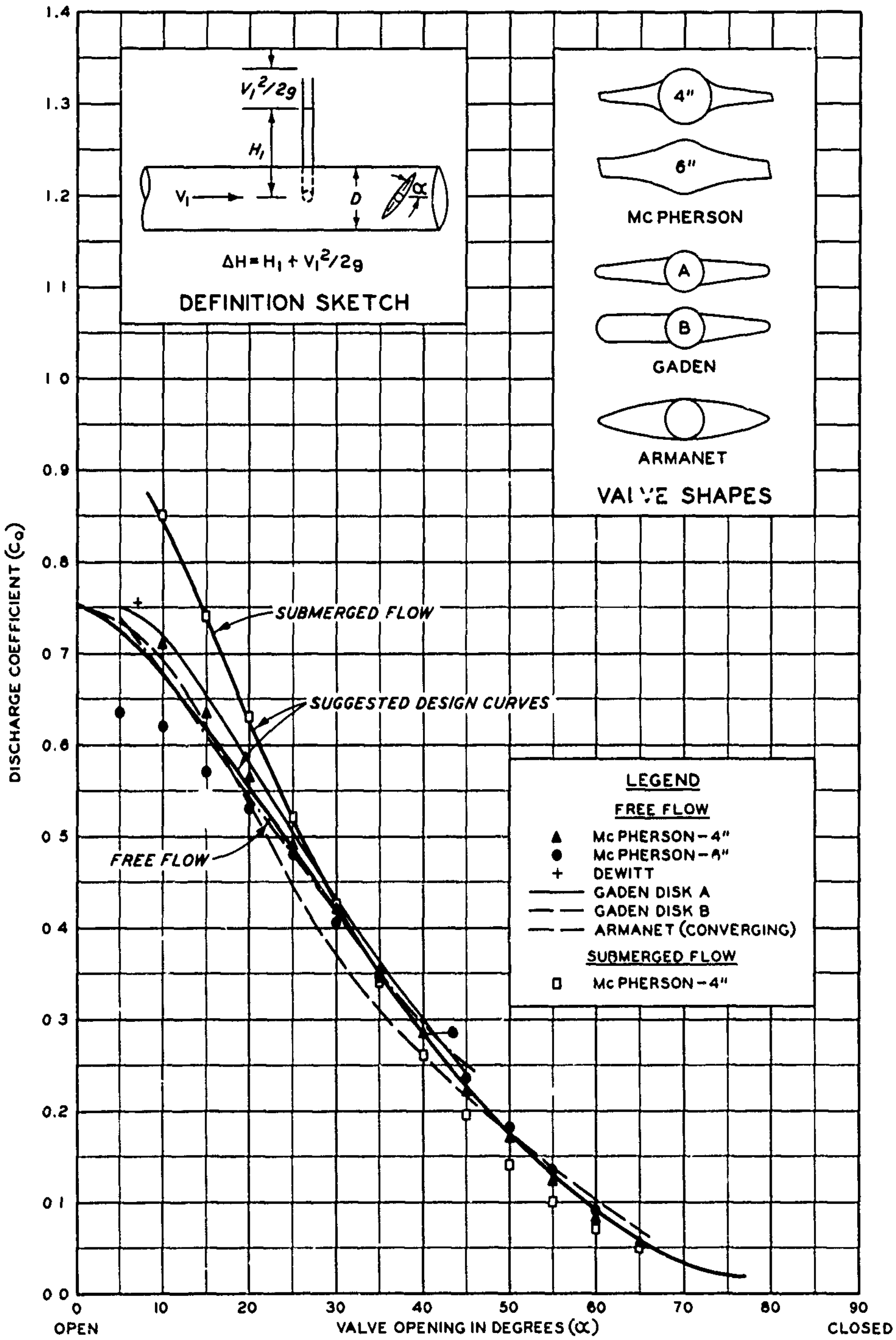
- Q = DISCHARGE IN CFS
- C_Q = DISCHARGE COEFFICIENT
- D = VALVE DIAMETER IN FT
- g = GRAVITY CONSTANT = 32 2 FT/SEC^2
- ΔH = PRESSURE DROP ACROSS THE VALVE IN FT OF WATER

BUTTERFLY VALVES

DISCHARGE COEFFICIENTS VALVE IN PIPE

HYDRAULIC DESIGN CHART 331-1

WES 8-58



BASIC EQUATION

$$Q = C_Q D^2 \sqrt{g}\sqrt{\Delta H}$$

WHERE

Q = DISCHARGE IN CFS

C_Q = DISCHARGE COEFFICIENT

D = VALVE DIAMETER IN FT

g = GRAVITY CONSTANT = 32.2 FT/SEC^2

ΔH = TOTAL ENERGY HEAD IN FT OF WATER UPSTREAM OF VALVE

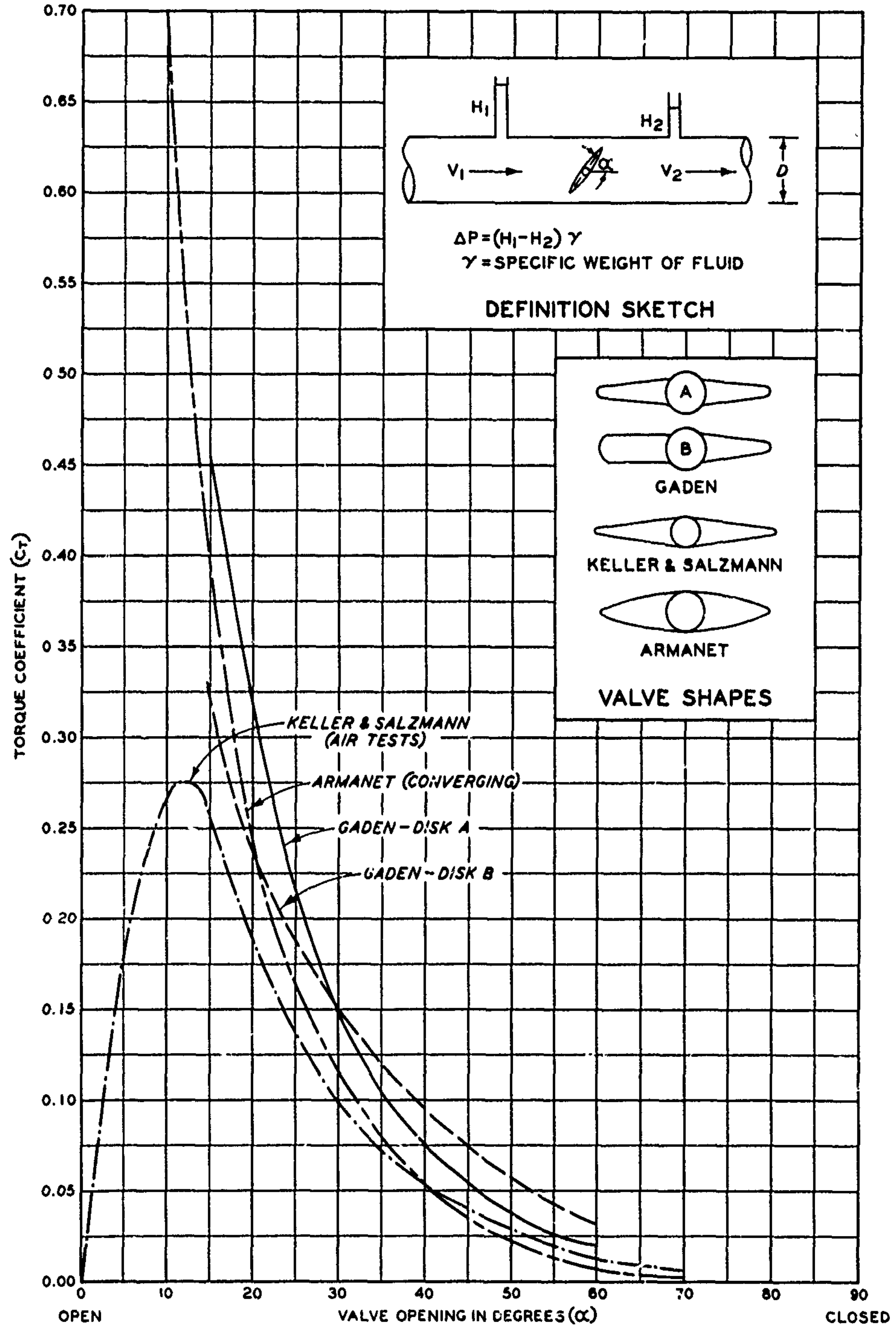
BUTTERFLY VALVES

DISCHARGE COEFFICIENTS

VALVE IN END OF PIPE

HYDRAULIC DESIGN CHART 331-1/1

WES 8-58



BASIC EQUATION

$$T = C_T D^3 \Delta P$$

WHERE

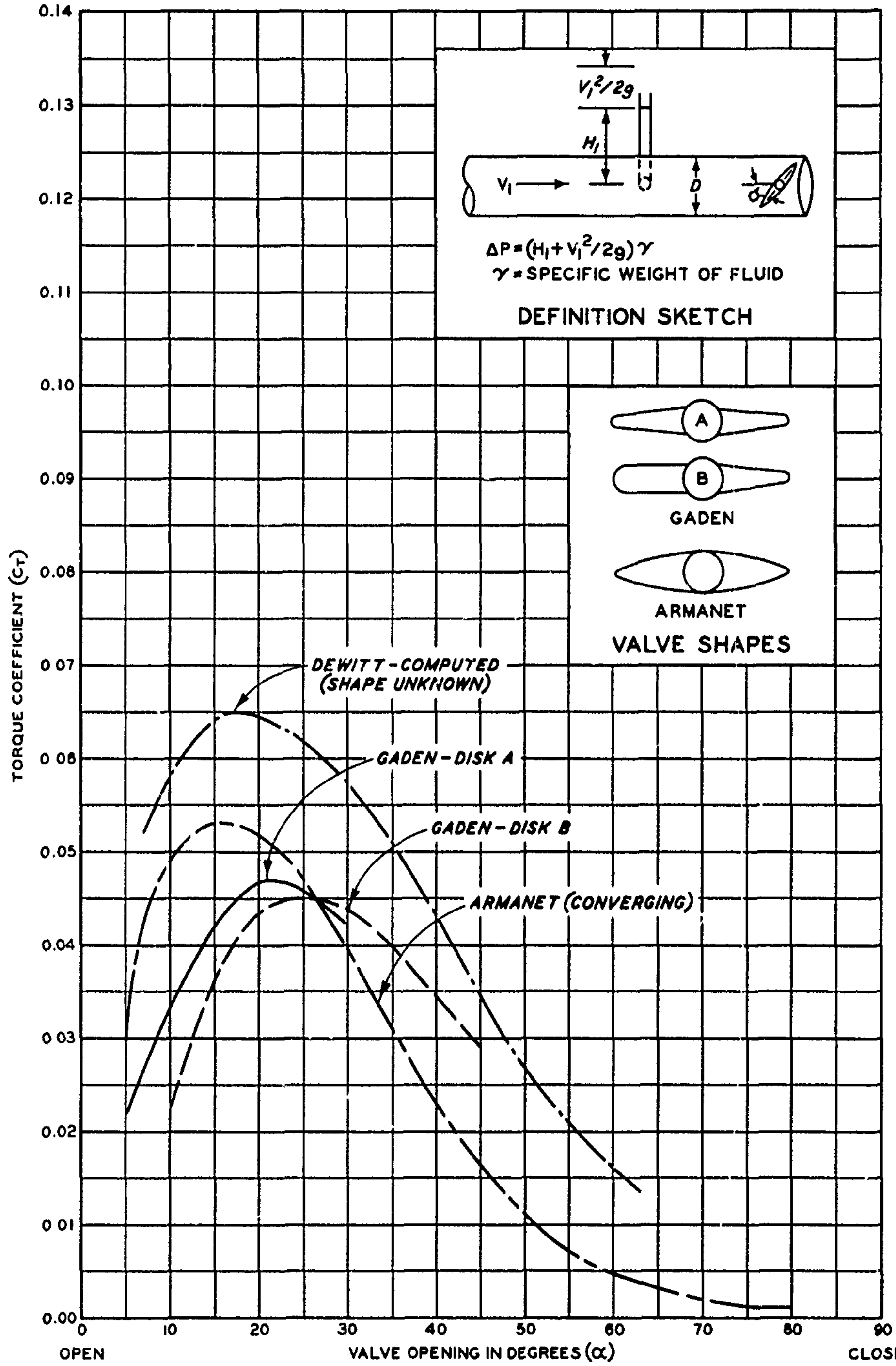
- T = TORQUE IN FT-LB
- C_T = TORQUE COEFFICIENT
- D = VALVE DIAMETER IN FT
- ΔP = PRESSURE DIFFERENTIAL IN LB/SQ FT

BUTTERFLY VALVES

TORQUE COEFFICIENTS VALVE IN PIPE

HYDRAULIC DESIGN CHART 331-2

WES 8-58



BASIC EQUATION

$$T = C_T D^3 \Delta P$$

WHERE:

- T = TORQUE IN FT-LB
- C_T = TORQUE COEFFICIENT
- D = VALVE DIAMETER IN FT
- ΔP = TOTAL ENERGY HEAD AT UPSTREAM SIDE OF VALVE IN LB/SQ FT

BUTTERFLY VALVES

TORQUE COEFFICIENTS VALVE IN END OF PIPE

HYDRAULIC DESIGN CHART 331-2/1

WES 8-58

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB CW 804 PROJECT John Doe Dam SUBJECT Butterfly Valves

COMPUTATION Valve Opening and Hydraulic Torque

COMPUTED BY WCB DATE 2/26/58 CHECKED BY RGC DATE 2/27/58

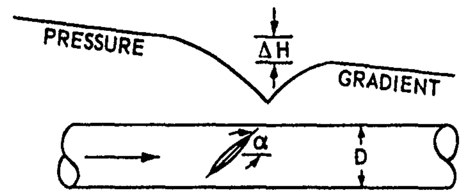
GIVEN:

Total available head (H_T) = 225 ft

Valve diameter (D) = 4 ft

Valve shape - Gaden-Disk A on Chart 331-1

Energy loss in system without valve (H_L) = 0.3 $V^2/2g$



ASSUME:

Discharge (Q) = 600 cfs

COMPUTE:

1. Head loss (H_L) in system without valve

$$V = \frac{Q}{A} = 48 \text{ ft per sec}$$

$$H_v = V^2/2g = 35 \text{ ft}$$

$$H_L = 0.3\, H_v = 10 \text{ ft}$$

2. Required valve loss (ΔH) for Q = 600 cfs

$$\Delta H = H_T - H_L - H_v = 225 - 10 - 35 = 180 \text{ ft}$$

Discharge coefficient (C_Q)

$$Q = C_Q D^2 \sqrt{g}\sqrt{\Delta H} \quad \text{(Chart 331-1)}$$

$$C_Q = \frac{600}{16 \times \sqrt{32.2} \times \sqrt{180}} = 0.49$$

From suggested design curve on Chart 331-1, valve opening (α) = 36° for C_Q of 0.49.

3. Hydraulic torque (T) for Q = 600 cfs and α = 36°. From Chart 331-2, torque coefficient (C_T) for Gaden-Disk A valve open 36° = 0.10.

$$T = C_T D^3 \Delta P \quad \text{(Chart 331-2)}$$

Where $\Delta P = (H_1 - H_2)\gamma = \Delta H \gamma$

$$T = 0.10 \times 64 \times 180 \times 62.5 = 72{,}000 \text{ ft-lb}$$

Repeat computations for other assumed discharges to determine discharge and hydraulic torque curves.

BUTTERFLY VALVES

SAMPLE COMPUTATION
DISCHARGE AND TORQUE

HYDRAULIC DESIGN CHART 331-3

WES 8-58

HYDRAULIC DESIGN CRITERIA

SHEETS 332-1 AND 1/1

HOWELL-BUNGER VALVES

DISCHARGE COEFFICIENTS

1. General. The Howell-Bunger valve is essentially a cylinder gate mounted with the axis horizontal. A conical end piece with its apex upstream is connected to the valve body by vanes. A movable external horizontal sleeve controls the discharge by varying the opening between the sleeve and the cone. The discharge is in the form of a diverging hollow conical jet. Diameters of valves range from 1.5 to 9 ft. Some valves have four vanes while others have six vanes. Separate discharge coefficient charts are presented for four- and six-vane valves.

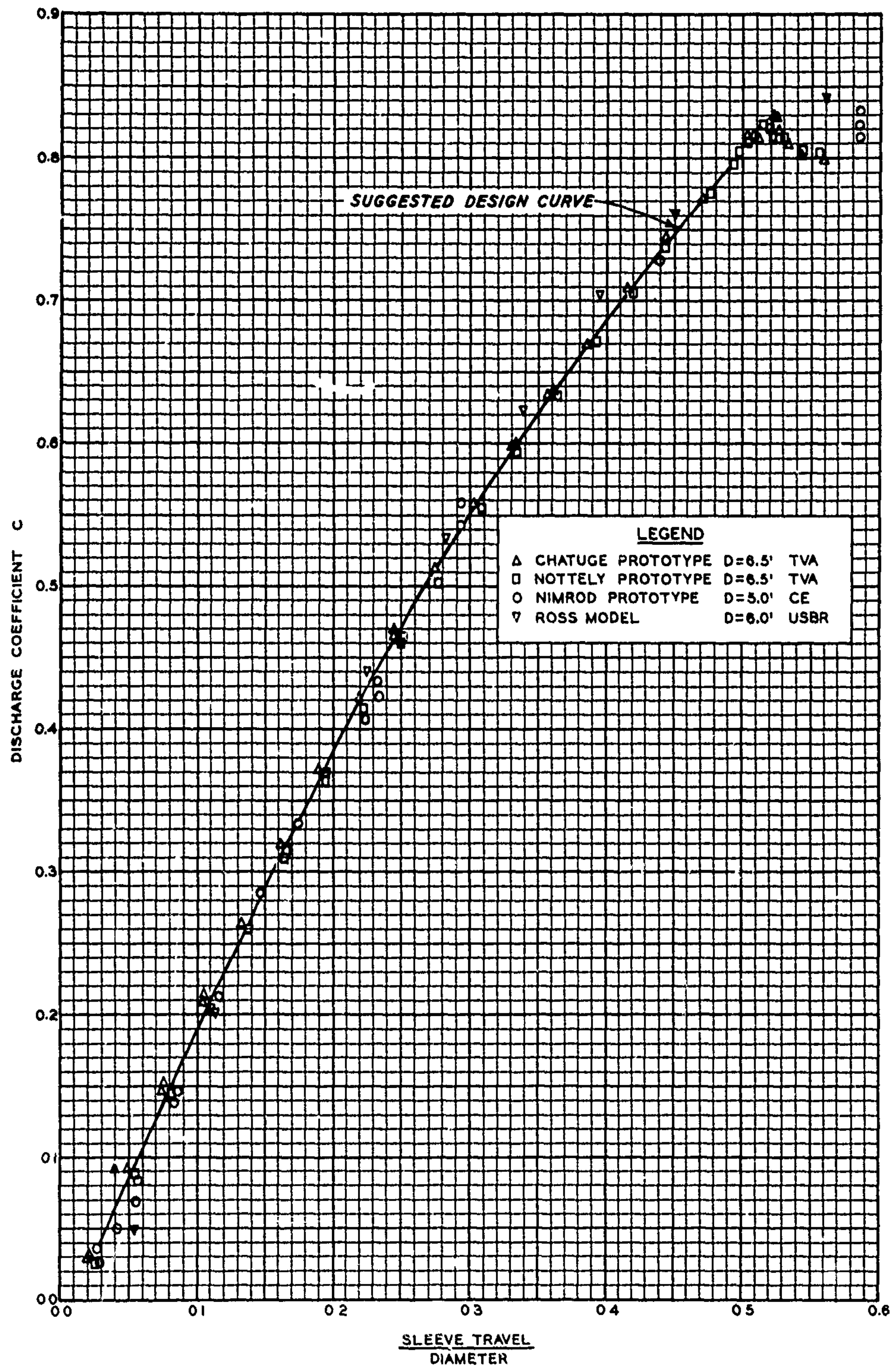
2. Discharge Coefficients. Discharge coefficients for Howell-Bunger valves have been computed for various dimensional features of the valves. However, the discharge coefficients shown on Charts 332-1 and 1/1 are based on the area of the conduit immediately upstream from the valve. The basic equation used is shown on each chart. The computed coefficients are plotted against the dimensionless factor, sleeve travel divided by conduit diameter.

3. Experimental Data. Discharge coefficients for Chatuge, Nottely, Watauga, and Fontana Dams were computed from prototype data published by the Tennessee Valley Authority[1]. Coefficients for Ross Dam are based on model data published by the Bureau of Reclamation[2]. Coefficients for Nimrod Dam result from discharge measurements made by the Little Rock District, CE. Coefficients for Narrows Dam result from model data obtained by the Waterways Experiment Station. The data presented on Charts 332-1 and 332-1/1 indicate discharge coefficients of 0.82 and 0.87 for full openings of the four- and six-vane valves, respectively.

---

(1) R. A. Elder and G. B. Dougherty, "Hydraulic Characteristics of Howell-Bunger Valves and Their Associated Structures," TVA Report dated 1 Nov. 1950.

(2) "Investigation of Hydraulic Properties of the Revised Howell-Bunger Valve, City of Seattle, Washington," Hydraulic Laboratory Report No. 168, Bureau of Reclamation, April 1945.



BASIC EQUATION

$$Q = CA\sqrt{2gH_e}$$

WHERE

C = DISCHARGE COEFFICIENT

A = AREA OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE IN SQ FT

H_e = ENERGY HEAD MEASURED TO CENTERLINE OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE IN FT

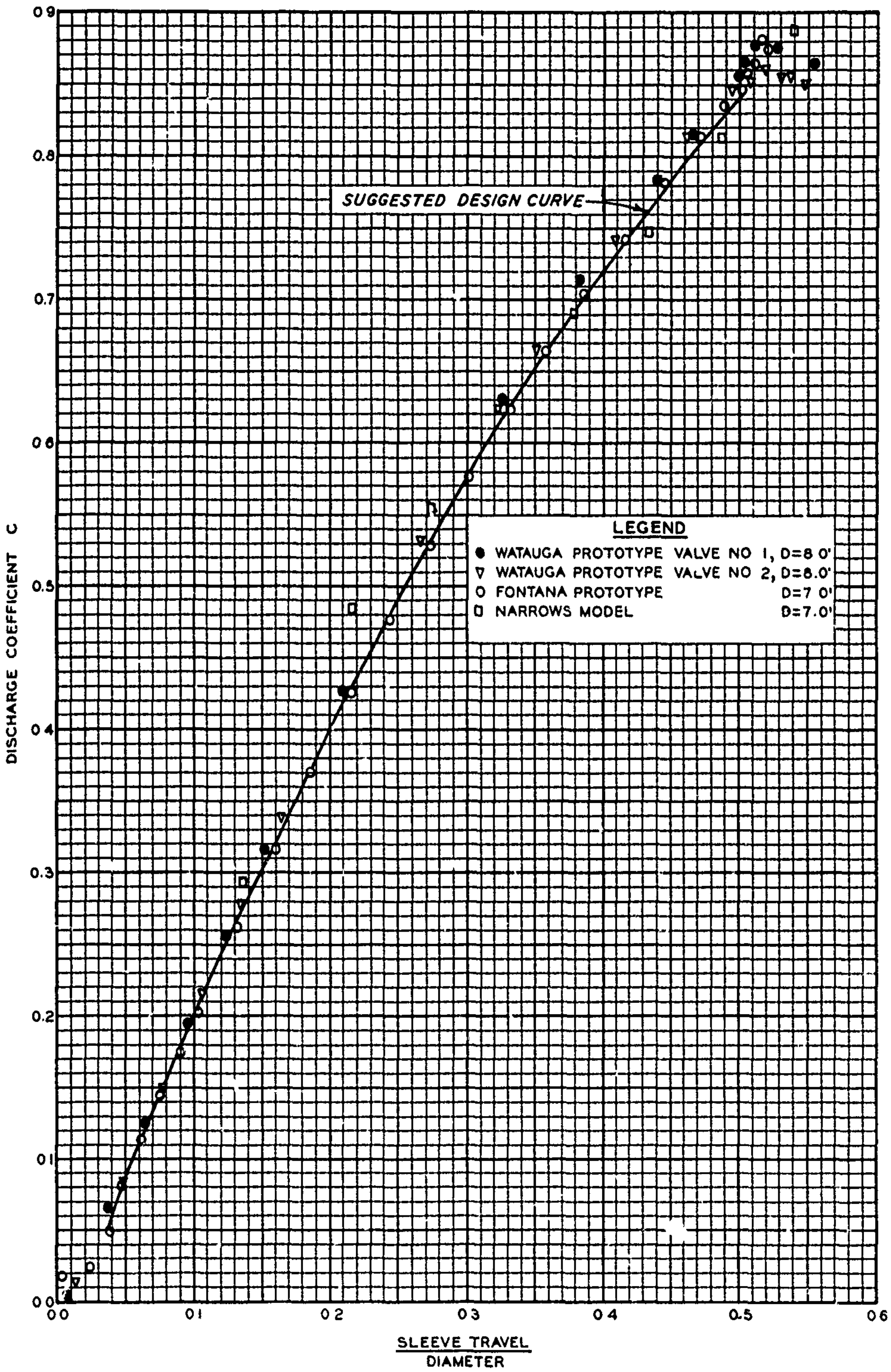
HOWELL - BUNGER VALVES

DISCHARGE COEFFICIENTS

FOUR VANES

HYDRAULIC DESIGN CHART 332-1

WES 2-54



BASIC EQUATION

$$Q = CA\sqrt{2gH_e}$$

WHERE

C = DISCHARGE COEFFICIENT

A = AREA OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE IN SQ FT

H_e = ENERGY HEAD MEASURED TO CENTERLINE OF CONDUIT IMMEDIATELY UPSTREAM FROM VALVE IN FT

HOWELL - BUNGER VALVES

DISCHARGE COEFFICIENTS

SIX VANES

HYDRAULIC DESIGN CHART 332-1/1

WES 2-54

SHEET 340-1

FLAP GATES

HEAD LOSS COEFFICIENTS

1. Flap gate head losses can be determined by the equation:

$$H_L = K \frac{V^2}{2g}$$

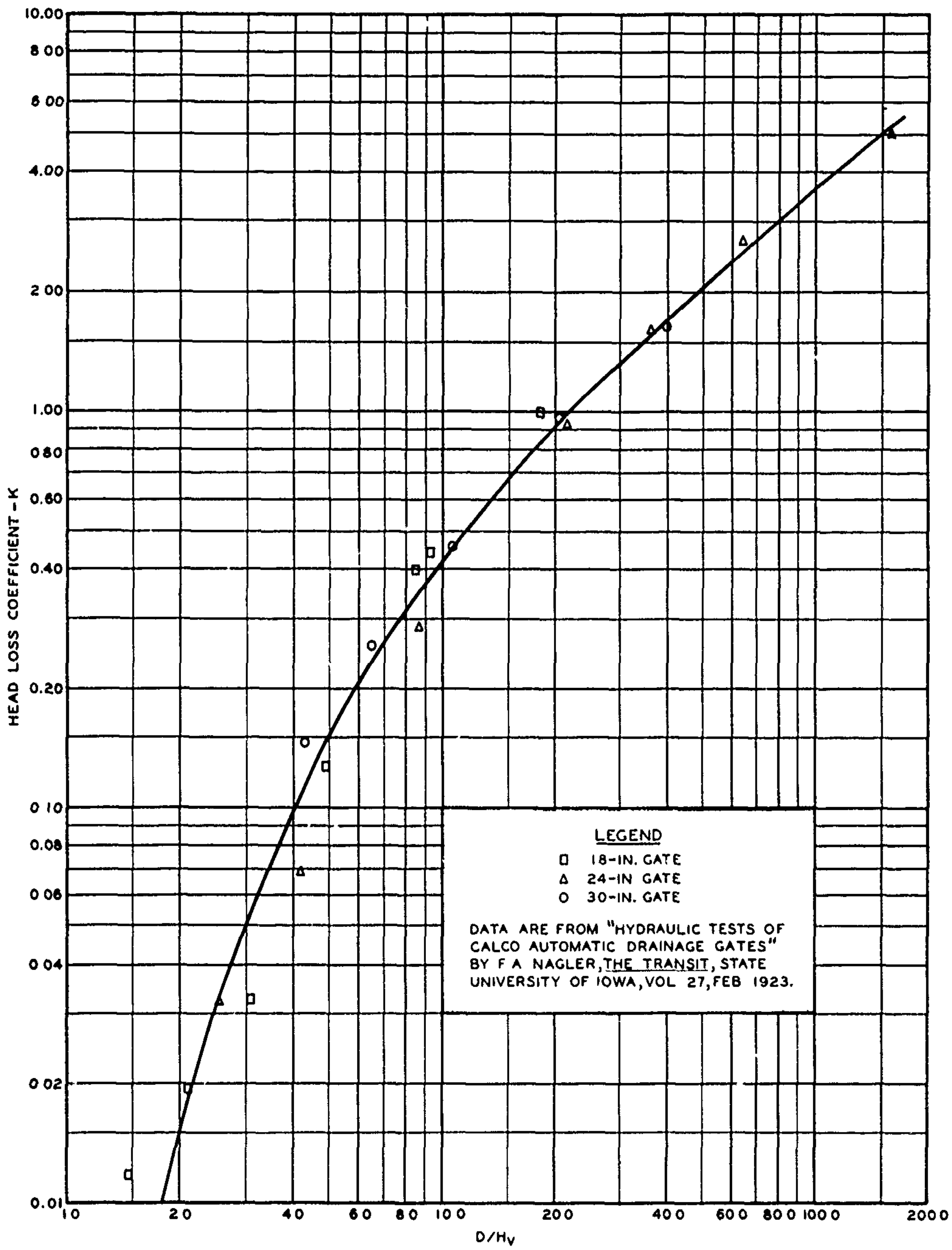
where

H_L = head loss in ft of water
K = head loss coefficient
V = conduit velocity in ft per sec

2. Hydraulic Design Chart 340-1 presents head loss coefficients for submerged flap gates. The data result from tests by Nagler (1) on 18-in.-, 24-in.-, and 30-in.-diameter gates.

3. Modern flap gates are heavier but similar in design to those tested by Nagler. It is suggested that Chart 340-1 be used for design purposes for submerged flow conditions until additional data become available. Head loss coefficient data are not available for free discharge.

(1) F. A. Nagler, "Hydraulic tests of Calco automatic drainage gates," The Transit, State University of Iowa, vol 27 (February 1923).



EQUATIONS

$$K = \frac{H_L}{H_V}, \quad H_V = \frac{V^2}{2g}$$

NOTE: K = HEAD LOSS COEFFICIENT
H_L = HEAD LOSS, FT
D = CONDUIT DIAMETER, FT
V = CONDUIT VELOCITY, FT/SEC
g = ACCELERATION OF GRAVITY, FT/SEC²

FLAP GATES
HEAD LOSS COEFFICIENTS
SUBMERGED FLOW
HYDRAULIC DESIGN CHART 340-1

HYDRAULIC DESIGN CRITERIA

SHEET 534-1

LOCK CULVERTS

REVERSE TAINTER VALVES

LOSS COEFFICIENTS

1. The head loss across a lock culvert valve can be determined from the equation:

$$H_L = K_v V^2/2g$$

where

H_L = head loss across the valve in ft of water
K_v = valve loss coefficient
V = mean culvert velocity in ft/sec
g = acceleration of gravity in ft/sec^2.

2. Hydraulic Design Chart 534-1 shows valve loss coefficients vs the ratio of the area of the valve opening to the area of the culvert for reverse tainter valves. The Weisbach curve(1) is based on data for a vertical gate in a rectangular conduit. The data shown were computed from model and prototype tests. A complete list of data sources is given in paragraph 3. The graph is similar to plate 6 of Engineer Manual 1110-2-1604. However, experimental data are plotted on Chart 534-1, to emphasize the excellent agreement of various test results.

3. Data Sources.

(1) Weisbach. "Hydraulics and Its Application" by A. H. Gibson, D. Van Nostrand Co., Inc., New York, N. Y., 4th ed., 1930, p 249.

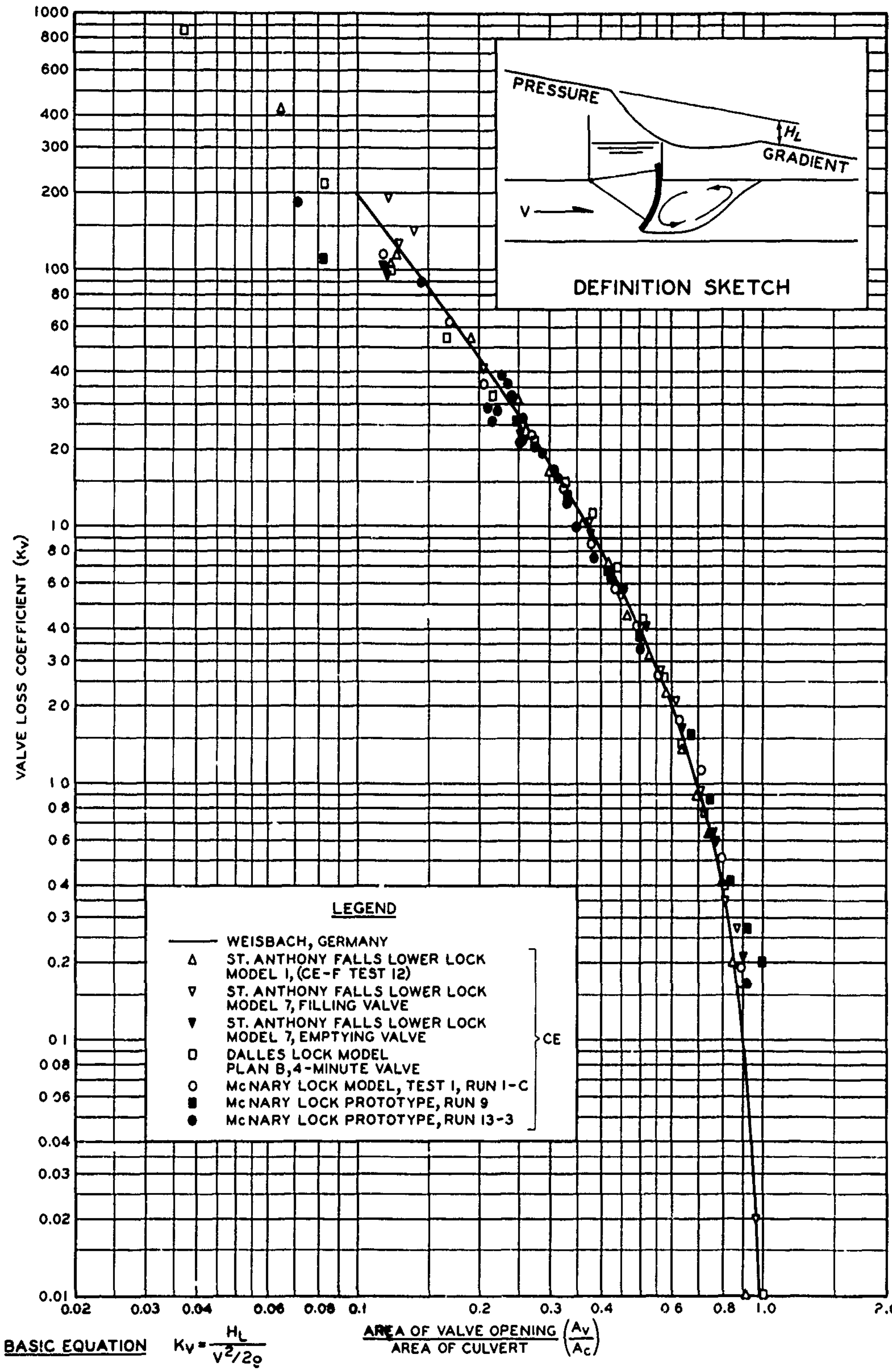
(2) St. Anthony Falls Lower Lock Models 1 and 7. Unpublished data computed by U. S. Army Engineer District, St. Paul, Minnesota, under CW 820, December 1953.

(3) McNary Lock Model, Test 1, Run 1-C. Unpublished data computed by U. S. Army Engineer District, St. Paul, Minnesota, under CW 820, December 1953.

(4) McNary Lock Prototype, Run 13-3. Report on Model-Prototype Conformity-McNary Dam Navigation Lock, 1955 Tests. U. S. Army Engineer District, Walla Walla, Washington, March 1959.

(5) McNary Lock Prototype, Run 9. Unpublished data computed by U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., from November 1957 tests.

(6) Dalles Lock Model. Report on Model-Prototype Conformity-McNary Dam Navigation Lock, 1955 Tests. U. S. Army Engineer District, Walla Walla, Washington, March 1959.



BASIC EQUATION $K_V = \dfrac{H_L}{V^2/2g}$

WHERE

K_V = VALVE LOSS COEFFICIENT

H_L = HEAD LOSS ACROSS VALVE IN FT OF WATER

V = AVERAGE VELOCITY IN FT/SEC

g = ACCELERATION OF GRAVITY-FT/SEC^2

LOCK CULVERTS

REVERSE TAINTER VALVES LOSS COEFFICIENT

HYDRAULIC DESIGN CHART 534-1

HYDRAULIC DESIGN CRITERIA

SHEETS 534-2 AND 534-2/1

LOCK CULVERTS

MINIMUM BEND PRESSURE

RECTANGULAR SECTION

1. Laboratory flow studies have shown that, for a rectangular conduit section, the minimum pressure in circular bends of 90 to 300 deg occurs on the inside of the bend 45 deg from the point of curvature. Experimental turbulent flow pressure data, at this location, closely approximate values computed for two-dimensional potential flow. McPherson and Strausser[1] have suggested an analytical procedure for determining the magnitude of the minimum pressure in a circular bend of rectangular section.

2. Theory. The minimum bend pressure head can be computed from the equation

$$C_p = \frac{H - H_i}{\frac{V^2}{2g}} \tag{1}$$

where

C_p = pressure-drop parameter

H = average pressure head, in ft, at the 45-deg point computed as a straight-line extension of the upstream pressure gradient

H_i = minimum pressure head, in ft, at the 45-deg point on inside of bend

V = average culvert velocity in ft per sec

g = acceleration, gravitational, in ft per sec^2

Equation 1 is similar to the bend coefficient equation developed by Lansford (reference 4, Sheet 228-3). Based on equation 3 of reference 1, it can also be shown that

$$C_p = \left[\frac{2}{\left(\frac{R}{C} - 1\right) \ln\left(\frac{\frac{R}{C} + 1}{\frac{R}{C} - 1}\right)}\right]^2 - 1 \tag{2}$$

where

R = center-line radius of the bend

C = one-half the culvert width

3. Application. Hydraulic Design Chart 534-2 shows the relation between the theoretical pressure-drop parameter and ratio of the radius of curvature to one-half the conduit dimension in the direction concerned. Values of C_p computed from experimental results reported by Silberman[2] and Yarnell and Woodward[3] are also shown. These data indicate the effects of Reynolds numbers between 6.7×10^4 and 8.2×10^5. Points computed from data summarized by McPherson and Strausser[1] from tests by Addison,[4] Lell,[5] Wattendorf,[6] and Nippert[7] and on the Waynesboro and Mt. Alto model studies at Lehigh University are included on the chart. The indicated Reynolds number is about 10^5 to 10^6. The chart is considered applicable to bends of 45 to 300 deg.

4. Cavitation occurs when the instantaneous pressure at any point in a flowing liquid drops to the vapor pressure. Vapor pressure varies with temperature of the liquid (see Sheet 000-2). Since turbulence in flow causes pressure fluctuations, an estimate should be made of the maximum expected fluctuation from the minimum computed bend pressure. The sum of the estimated pressure fluctuation, the vapor pressure, and a few feet of water for a margin of safety should be computed. The local barometric pressure (see Chart 000-2) should be subtracted from this total to obtain the minimum permissible bend pressure. This pressure can then be used to determine the necessary average conduit pressure or the permissible average conduit velocity to prevent cavitation. Cavitation damage has been found where the average pressure is relatively high but violent negative pulsations reach cavitation pressures. Such criteria as indicated here should therefore be used conservatively.

5. Chart 534-2/1 is a sample computation showing the application of Chart 534-2 to the minimum bend pressure problem. Computations to indicate the minimum permissible average conduit pressure and the maximum permissible average conduit velocity to prevent cavitation are included. Chart 534-2 can also be used for the design of bends in rectangular sluices and siphons and in circular conduits. Its application to the latter is shown in Chart 228-3.

6. References.

(1) McPherson, M. B., and Strausser, H. S., "Minimum pressures in rectangular bends." Proceedings, ASCE, vol 81, Separate Paper No. 747 (July 1955); vol 82, Separate Paper No. 1092 (October 1956), p 9, Closure.

(2) Silberman, E., The Nature of Flow in an Elbow. Project Report No. 5, St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, prepared for David Taylor Model Basin, December 1947.

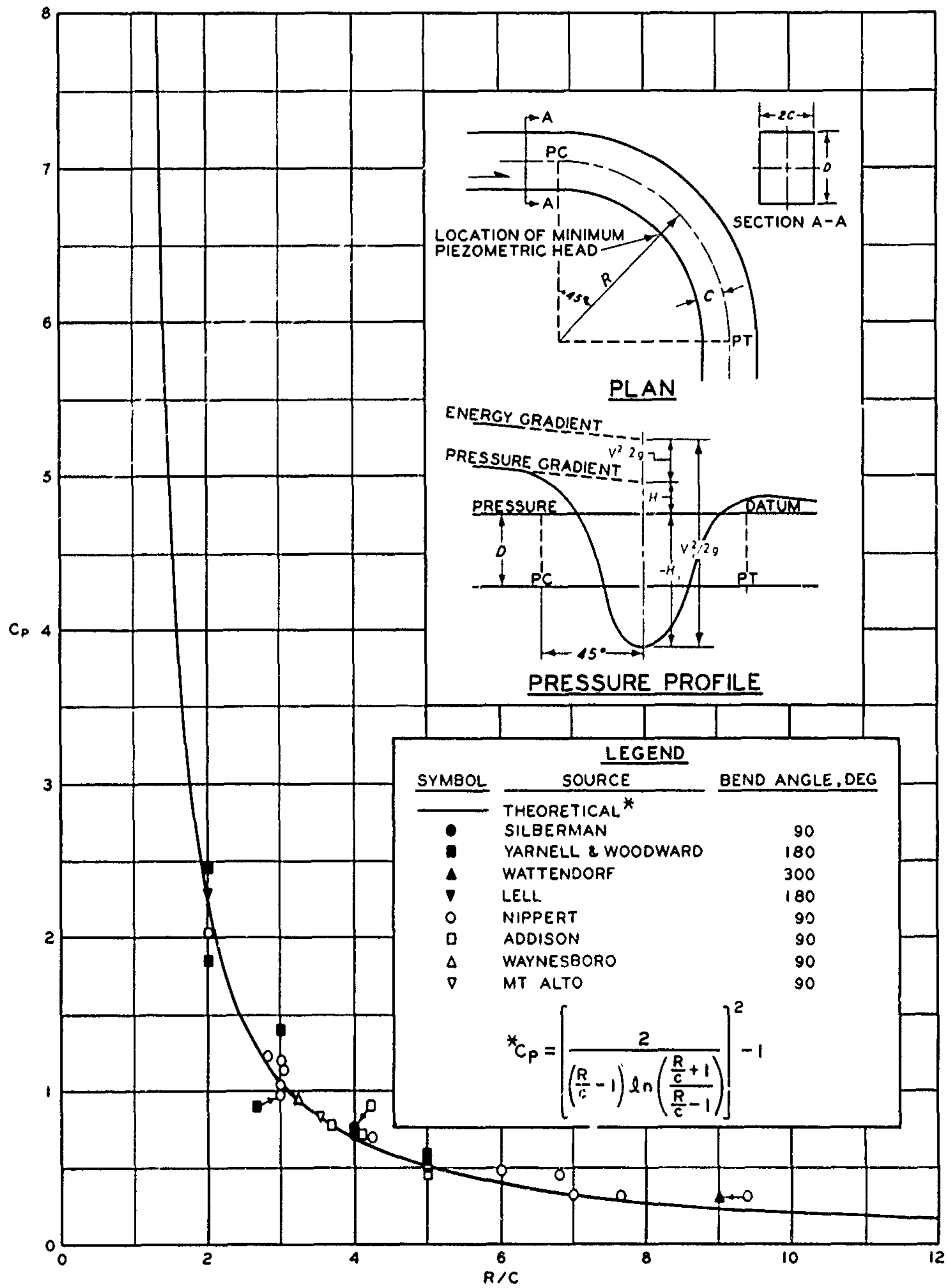
(3) U. S. Department of Agriculture, Flow of Water Around 180-Degree Bends, by D. L. Yarnell, and S. M. Woodward. Technical Bulletin No. 526, Washington, D. C., October 1936.

(4) Addison, H., "The use of bends as flow meters." Engineering, vol 145 (4 March 1938), pp 227-229 (25 March 1938), p 324.

(5) Lell, J., "Contribution to the Knowledge of Secondary Currents in Curved Channels (Beitrag zur Kenntnis der Sekundärströmungen in gekrümmten Kanälen)." Dissertation, R. Oldenbourg, Muchen, 1913. Also Zeitschrift für das gesamte Turbinenwesen, Heft 11, July 1914, pp 129-135, 293-298, 313-317, and 325-330.

(6) Wattendorf, F. L., "A study of the effects of curvature on fully developed turbulent flow." Proceedings, Royal Society of London, Series A, vol 148 (February 1935), pp 565-598.

(7) Nippert, H., "Über den Strömungsverlust in gekrümmten Kanälen." VDI, Forschungsarbeiten, Heft 320, Berlin (1929).



EQUATIONS

$$H+\frac{V^2}{2g}=H_1+\frac{V_1^2}{2g}\ , \qquad \frac{H-H_1}{\frac{V^2}{2g}}=C_p$$

WHERE:

- H = PIEZOMETRIC HEAD FROM PRESSURE GRADIENT EXTENSION, FT
- V = AVERAGE VELOCITY, FT PER SEC
- g = ACCELERATION, GRAVITATIONAL, FT PER SEC^2
- H_1 = MINIMUM PIEZOMETRIC HEAD, FT
- V_1 = VELOCITY AT LOCATION OF H_1, FT PER SEC
- C_p = PRESSURE DROP PARAMETER

LOCK CULVERTS
RECTANGULAR SECTION MINIMUM BEND PRESSURE
HYDRAULIC DESIGN CHART 534-2

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

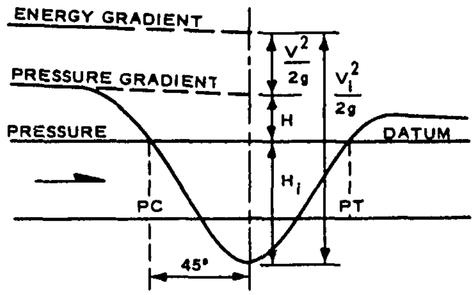
JOB CW 804 PROJECT John Doe Dam SUBJECT Lock Culverts

COMPUTATION Minimum Bend Pressure in a Rectangular Section

COMPUTED BY WTH DATE 4/30/59 CHECKED BY MBB DATE 5/4/59

GIVEN:

Rectangular culvert section
Horizontal bend
Elevation of roof = 500 ft msl
Deflection angle = 90°
Bend radius (R) = 10 ft
Width of culvert (2c) = 10 ft
Average velocity (V) = 20 fps
Temperature = 50 F
Average conduit pressure measured from pressure gradient extension (H) = 10 ft



PRESSURE PROFILE

REQUIRED:

H_i = minimum pressure (in ft) inside of bend.
$H_{i\,min}$ = minimum permissible bend pressure (ft).
H_{min} = minimum permissible average conduit pressure (in ft) to prevent cavitation (V = 20 fps).
V_{max} = maximum permissible average conduit velocity (in fps) to prevent cavitation (H = 10 ft).

COMPUTE:

1. $R/c = 10/5 = 2$
2. $C_p = 2.30$ for $R/c = 2$ (Chart 534-2)
3. Minimum bend pressure (H_i)

$$\frac{H - H_i}{V^2/2g} = C_p$$

$$\frac{10 - H_i}{20^2/64.4} = 2.30$$

$H_i = -4.3$ ft

4. Minimum permissible bend pressure head ($H_{i\,min}$)
   a. Estimated pressure head fluctuation = 10.0 ft
   b. Vapor pressure head of water at 50 F = 0.4 ft (Sheet 000-2)
   c. Pressure allowance for margin of safety = 5.0 ft
      Total = 15.4 ft
   d. Local barometric pressure head = 33.2 ft (Chart 000-2)
   e. Minimum permissible bend pressure head ($H_{i\,min}$) = 15.4 - 33.2 = -17.8 ft

Note: Since $H_i > H_{i\,min}$ cavitation should not occur. However, this is not adequate to use as positive criterion since the values used for items 4a and 4c are dependent upon the judgement of the designer.

5. Minimum permissible average conduit pressure head (H_{min}) to prevent cavitation (V = 20 fps).

$$\frac{H_{min} - H_{i\,min}}{V^2/2g} = C_p$$

$$\frac{H_{min} - (-17.8)}{20^2/64.4} = 2.30$$

$H_{min} = 2.3\,(400/64.4) - 17.8 = 14.3 - 17.8 = -3.5$ ft

6. Maximum permissible average conduit velocity (V_{max}) to prevent cavitation (conditions of step 4 and H = 10 ft).

$$\frac{H - H_{i\,min}}{V^2_{max}/2g} = C_p$$

$$\frac{10 - (-17.8)}{V^2_{max}/64.4} = 2.3$$

$$V^2_{max} = \frac{(10 + 17.8)\,64.4}{2.3} = \frac{27.8 \times 64.4}{2.3} = 779$$

LOCK CULVERTS
RECTANGULAR SECTION
MINIMUM BEND PRESSURE
SAMPLE COMPUTATION
HYDRAULIC DESIGN CHART 534-2/1

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

SHEETS 610-1 to 610-7

TRAPEZOIDAL CHANNELS

1. Hydraulic Design Charts 610-1 to 610-7 are design aids for reducing the computation effort in the design of trapezoidal channels having various side slopes from 1 to 1 to 3 to 1 with uniform subcritical or supercritical flow. It is expected that the charts will be of value in preliminary design work where different channel sizes, roughness values, and slopes are to be investigated. Certain features of the charts were based on graphs prepared by the Los Angeles District, CE. Charts 610-1 to 610-7 can be used to interpolate values for intermediate side slopes.

2. Basic Equations. Manning's formula for open channel flow,

$$Q = \frac{1.486\ A\ S^{1/2}\ R^{2/3}}{n}$$

can be separated into a factor, involving slope and friction

$$C_n = \frac{1.486\ S^{1/2}}{n}$$

and a geometric factor involving area and hydraulic radius

$$C_k = AR^{2/3} .$$

Chart 610-1 and -1/1 show values of the factor, C_n , for slopes of 0.0001 to 1.0 and n values of 0.010 to 0.035. Charts 610-2 to -4/1-1 show values of the geometric factor, C_k , for base widths of 0 to 600 ft and depths of 2 to 30 ft. Charts 610-5 to -7 show values of critical depth divided by the base width for discharges of 1,000 to 200,000 cfs and base widths of 4 to 600 ft.

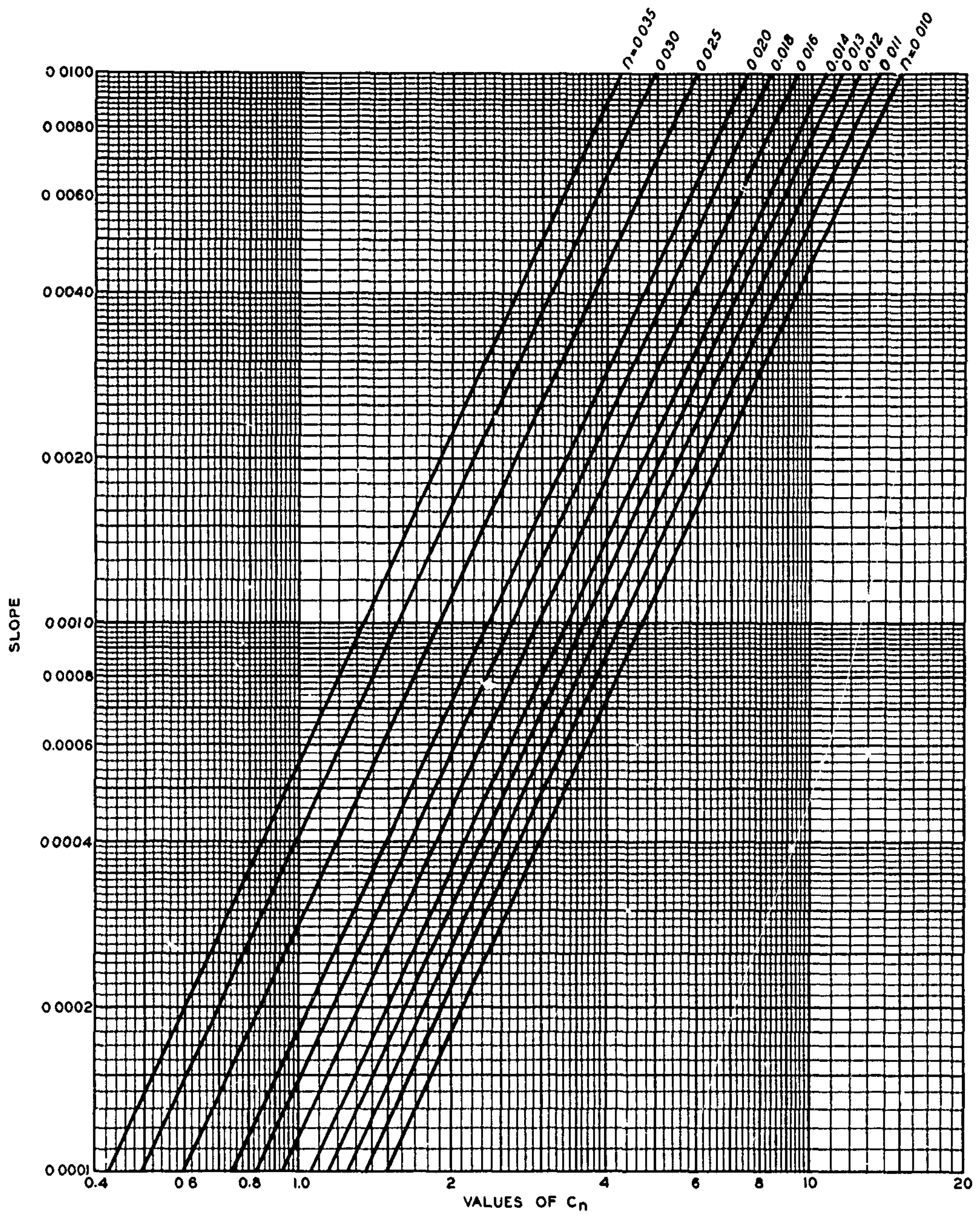
3. Application. Preliminary design of trapezoidal channels for subcritical or supercritical flow is readily determined by use of the charts in the following manner:

a. With given values of n and S , C_n can be obtained from charts 610-1 and -1/1.

b. Since $Q = C_n C_k$ the required value of C_k can be obtained by dividing the design Q by C_n .

c. With the required C_k value, suitable channel dimensions can be selected from charts 610-2 to -4/1-1.

d. Charts 610-5 to 610-7 can be used to determine the relation of design depth to critical depth.



FORMULA:

$$C_n = \frac{1.486\, S^{\frac{1}{2}}}{n}$$

WHERE
S = SLOPE
n = MANNING'S "n"

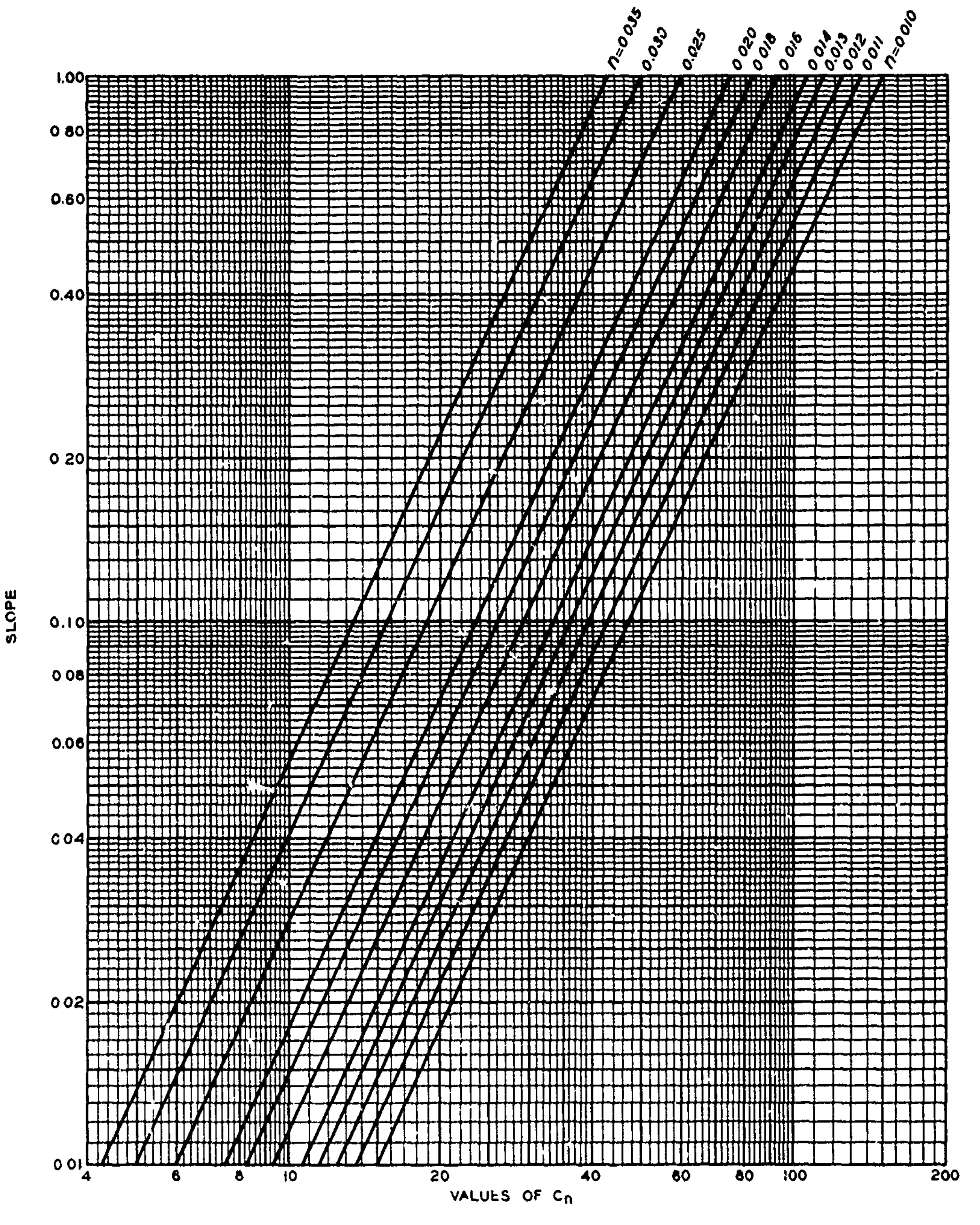
OPEN CHANNEL FLOW

SLOPE COEFFICIENTS

$0.0001 < S < 0.010$

HYDRAULIC DESIGN CHART 610-1

REVISED 8-58 WES 2-54



FORMULA:

$$C_n = \frac{1.486\, S^{\frac{1}{2}}}{n}$$

WHERE
S = SLOPE
n = MANNING'S "n"

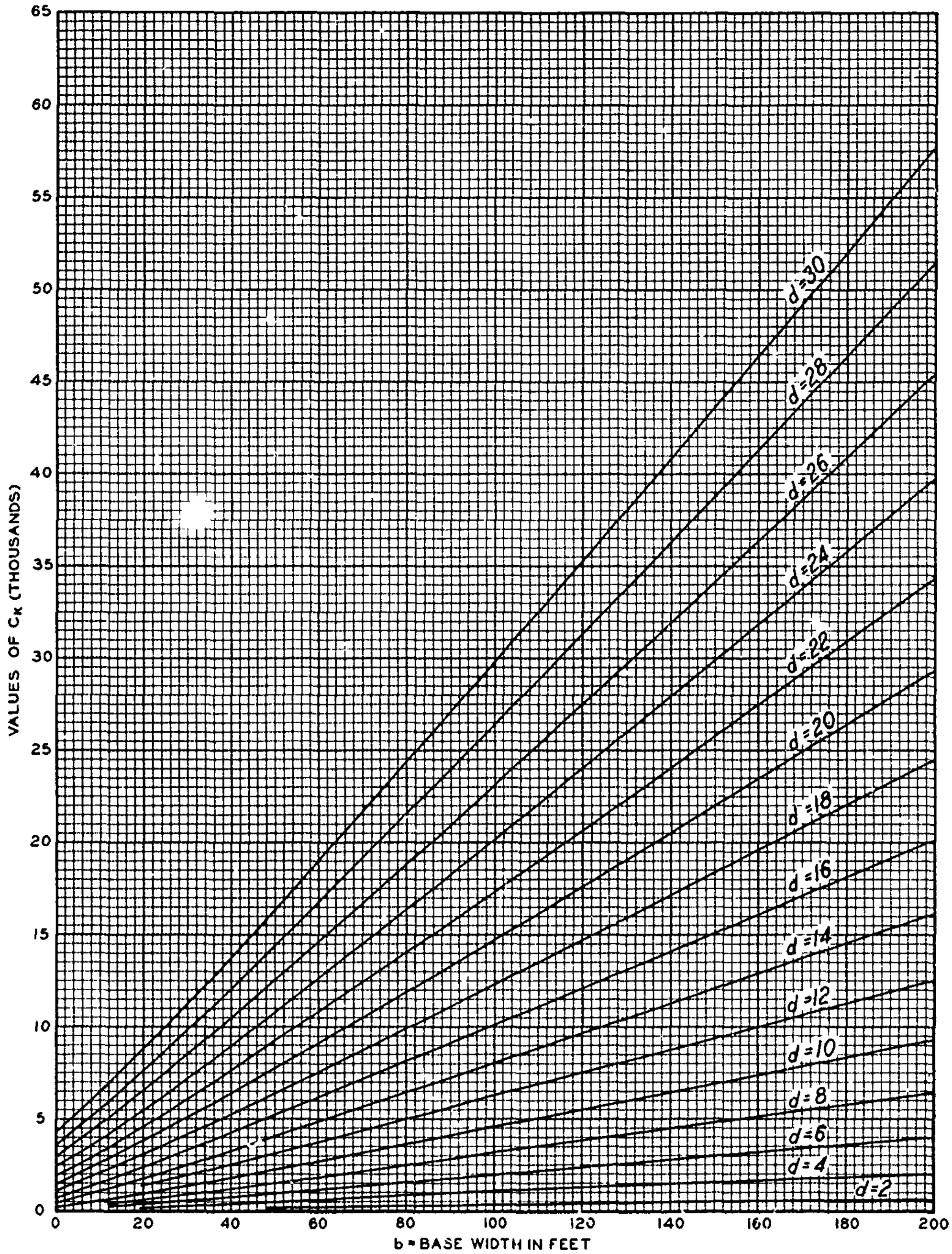
OPEN CHANNEL FLOW

SLOPE COEFFICIENTS

0.01 < S < 1.00

HYDRAULIC DESIGN CHART 610-1/1

REVISED 8-58 WES 2-54

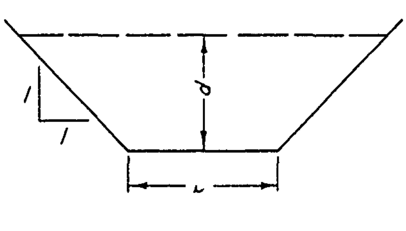


$C_K = AR^{2/3}$

WHERE

A = AREA

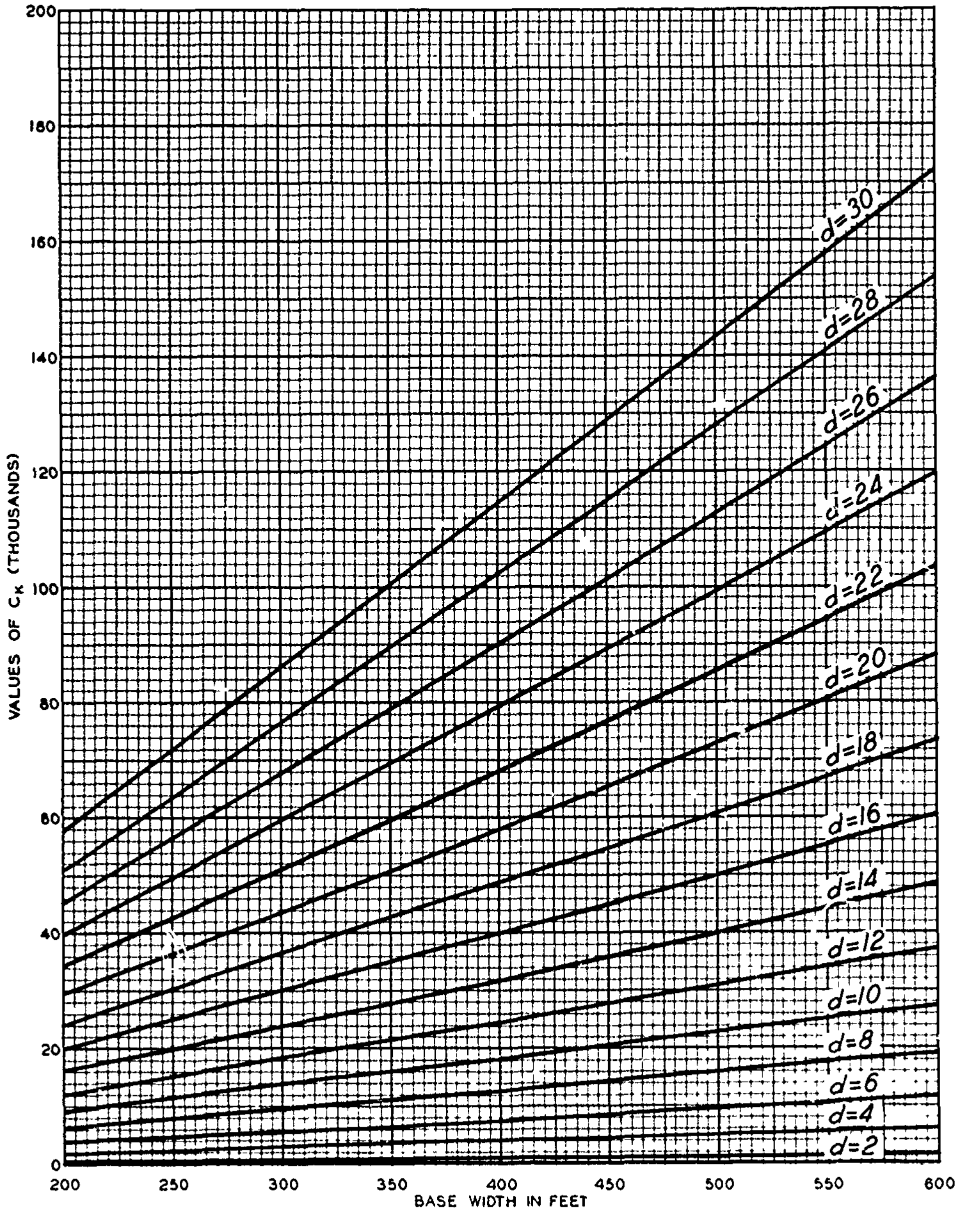
R = HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE 1 TO 1

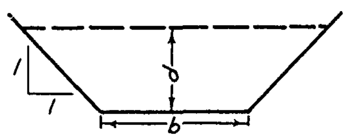
HYDRAULIC DESIGN CHART 610-2

WES 2-54



$C_K = AR^{2/3}$

WHERE
A=AREA
R=HYDRAULIC RADIUS

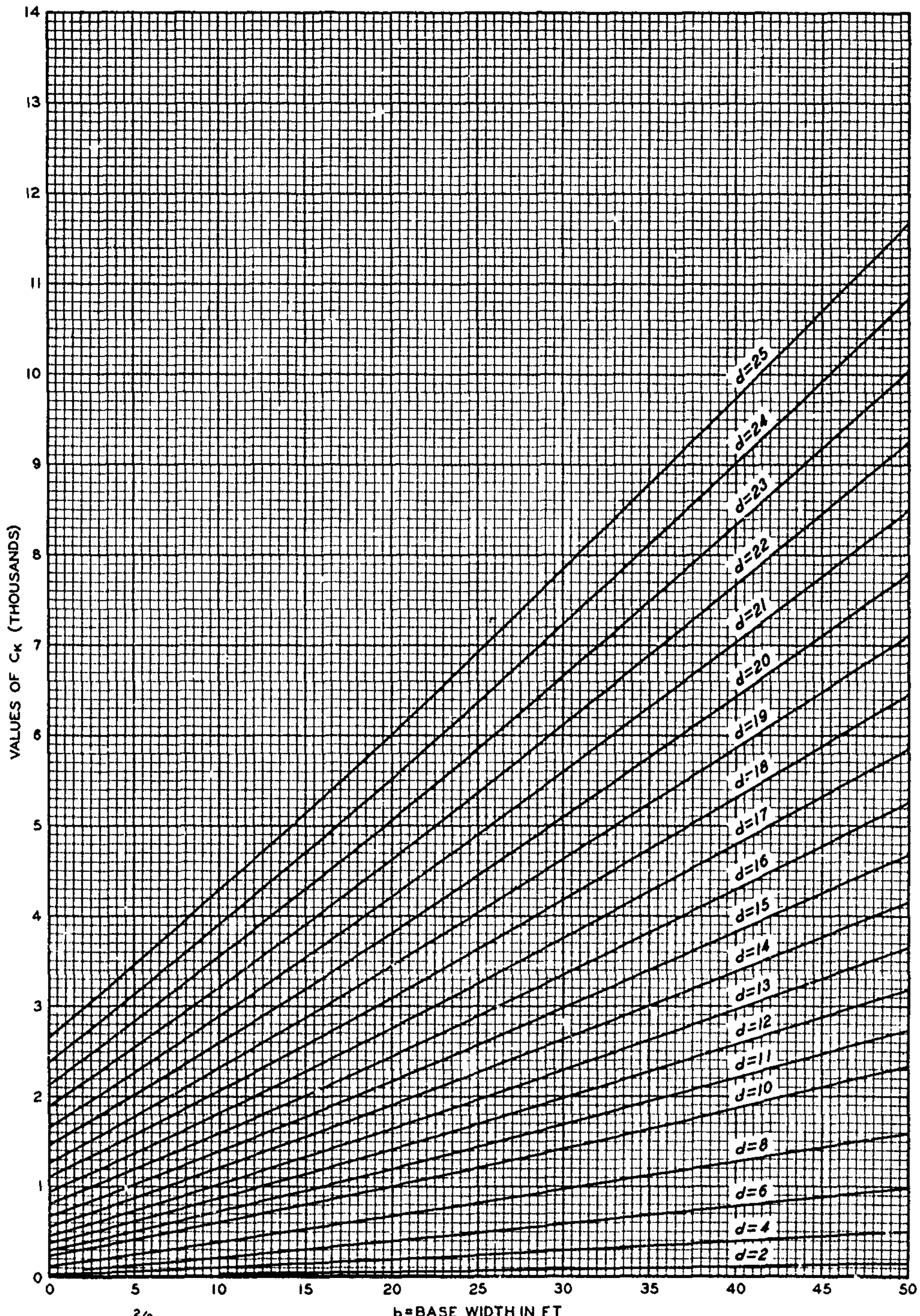


TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH

SIDE SLOPE 1 TO 1

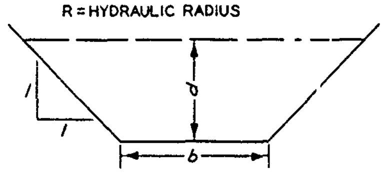
HYDRAULIC DESIGN CHART 610-2/1

WES 9-54



$C_K = AR^{2/3}$

WHERE
A = AREA
R = HYDRAULIC RADIUS

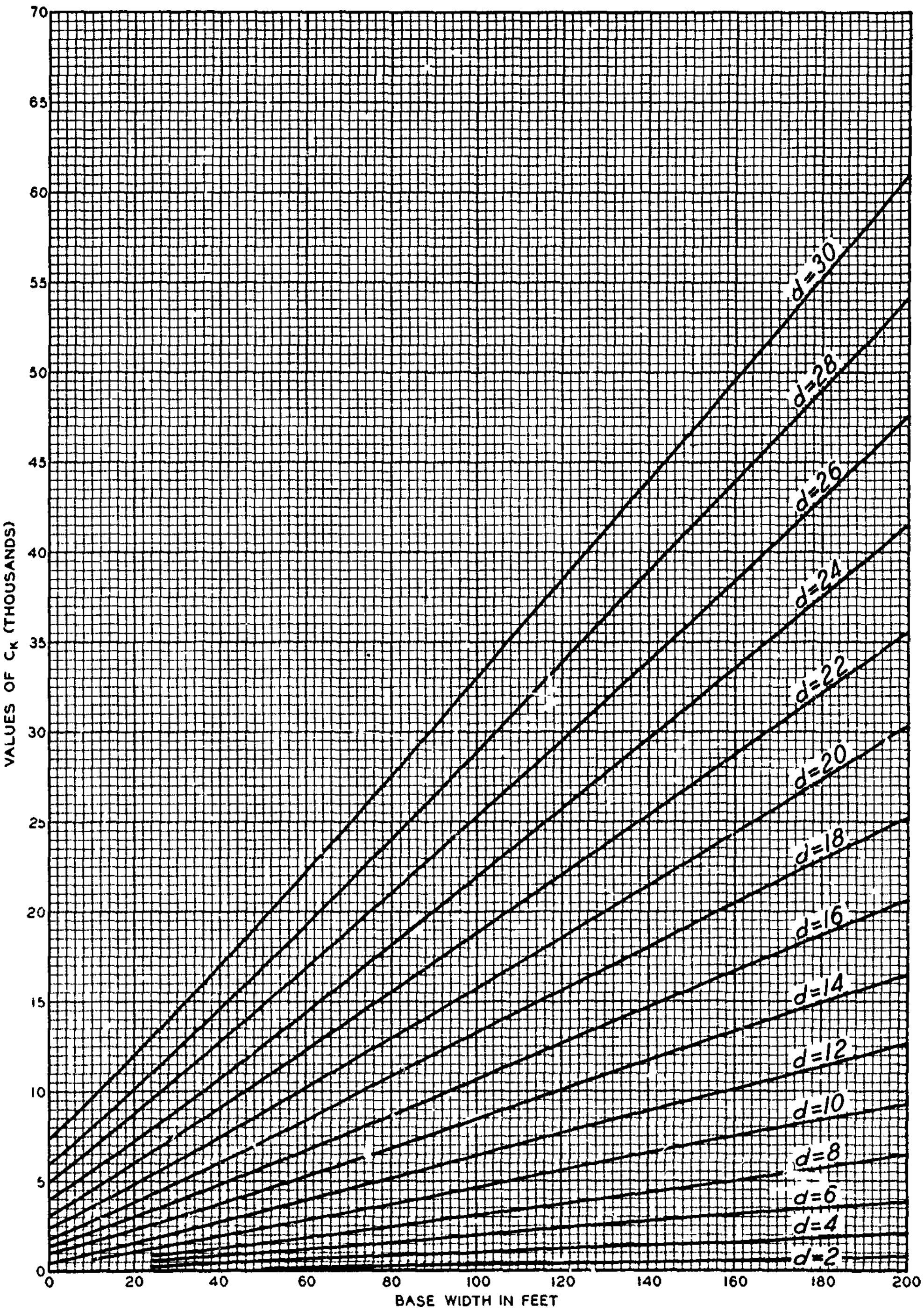


TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE 1 TO 1
BASE WIDTH 0 TO 50 FEET

HYDRAULIC DESIGN CHART 610-2/1-1

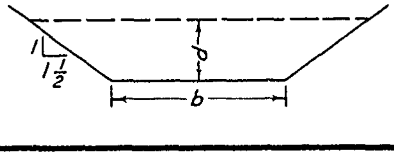
PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 5-59



$C_K = AR^{2/3}$

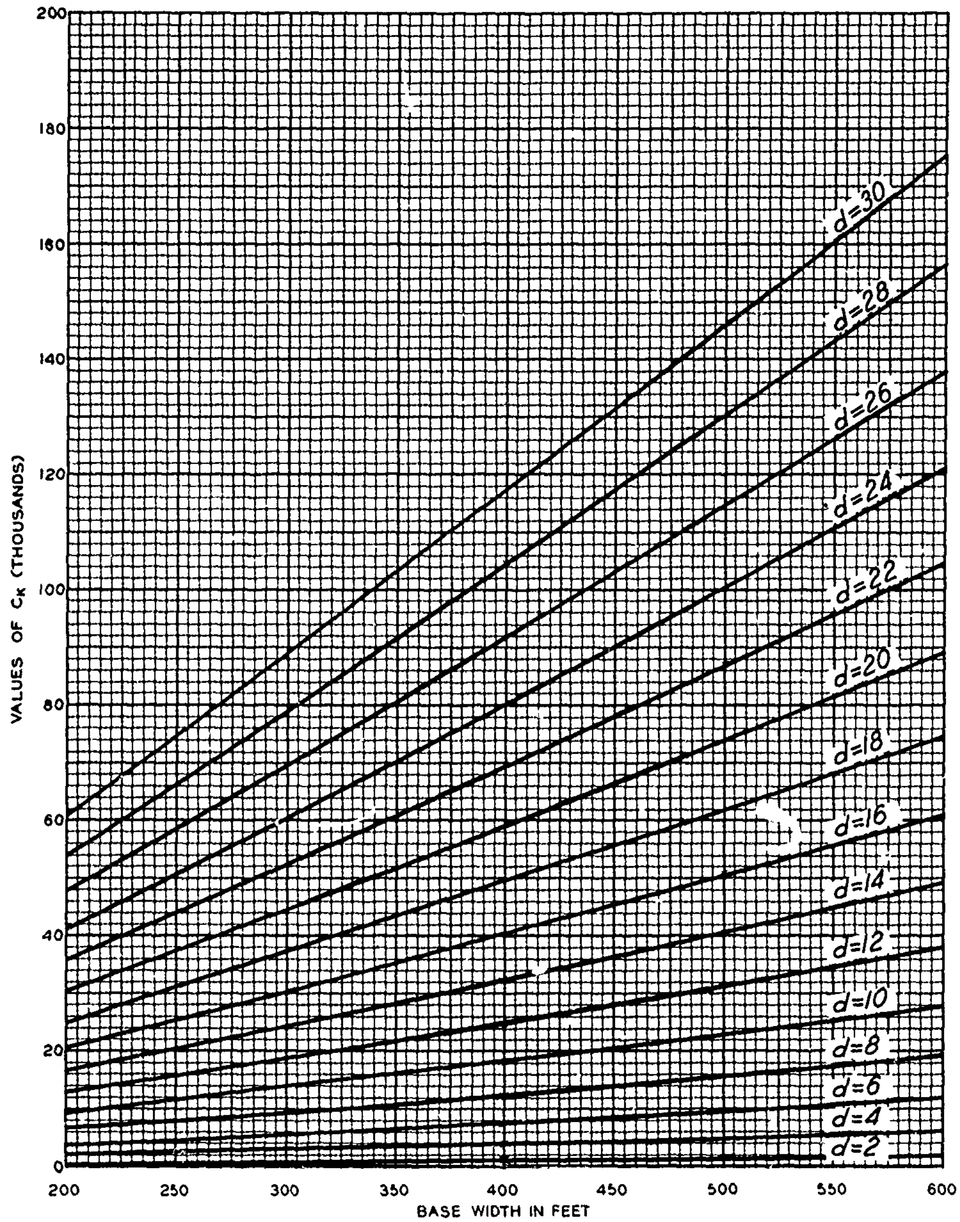
WHERE
A = AREA
R = HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $1\frac{1}{2}$ TO 1

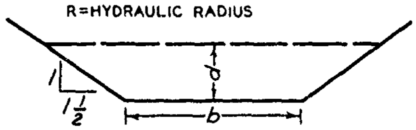
HYDRAULIC DESIGN CHART 610-2/2

WES 9-54



$C_K = AR^{2/3}$

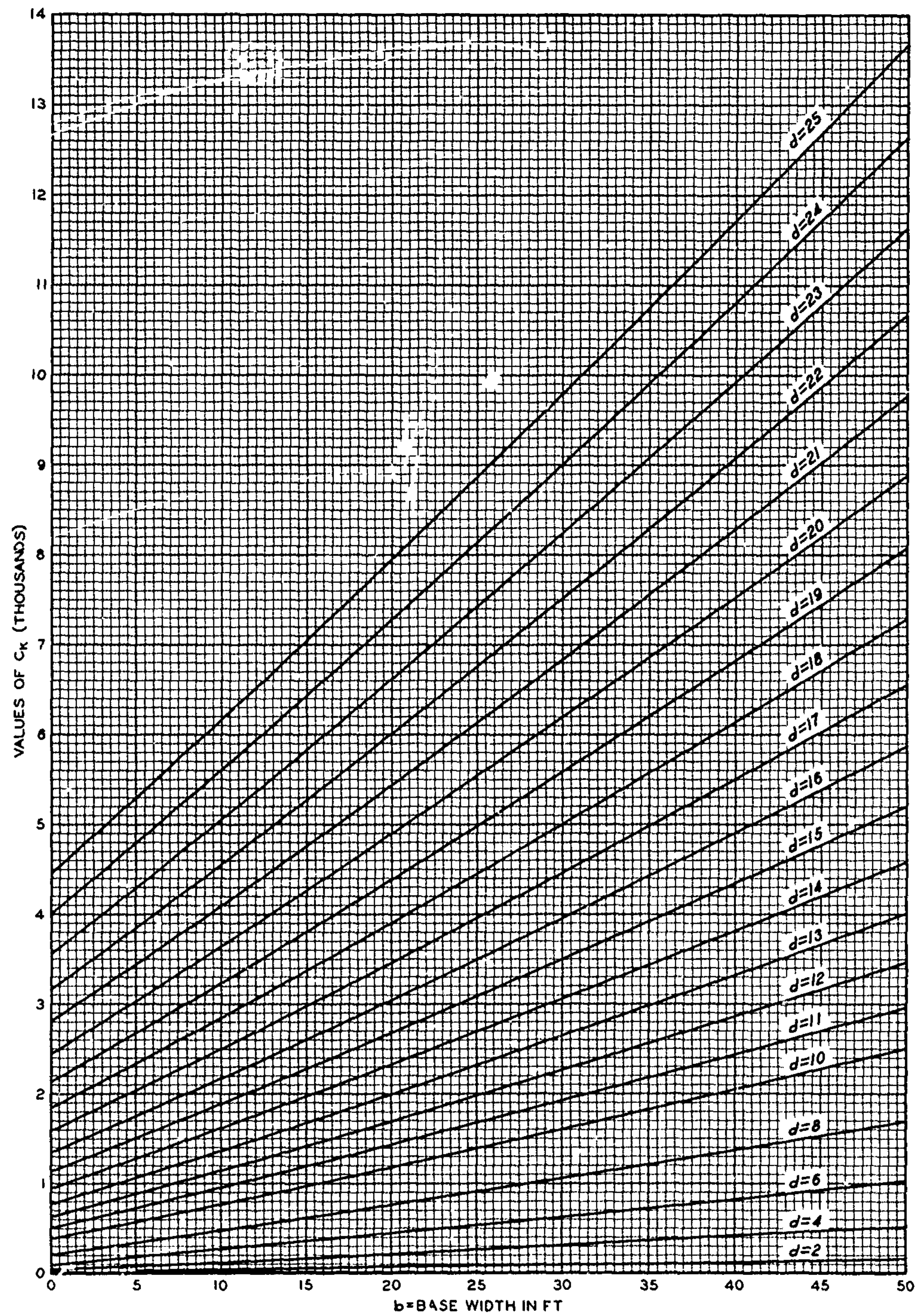
WHERE
A=AREA
R=HYDRAULIC RADIUS



TPAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $1\frac{1}{2}$ TO 1

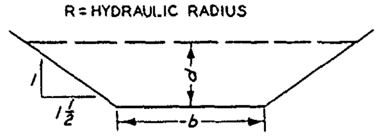
HYDRAULIC DESIGN CHART 610-2/3

WES 9-54



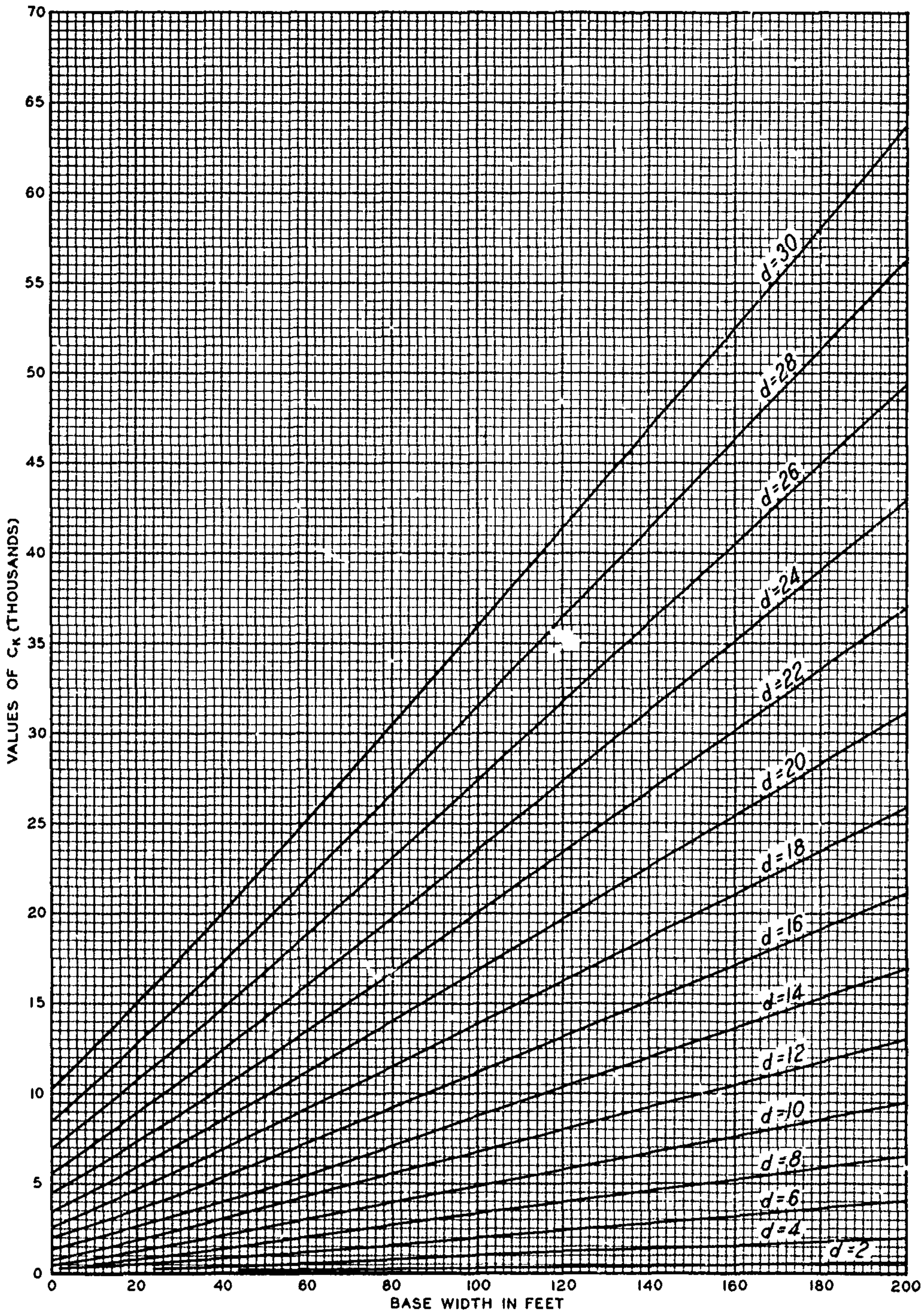
$C_K = AR^{2/3}$

WHERE
A = AREA
R = HYDRAULIC RADIUS



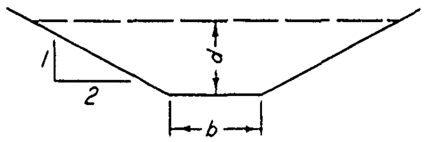
TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $1\frac{1}{2}$ TO 1
BASE WIDTH 0 TO 50 FEET

HYDRAULIC DESIGN CHART 610-2/3-1



$C_K = AR^{2/3}$

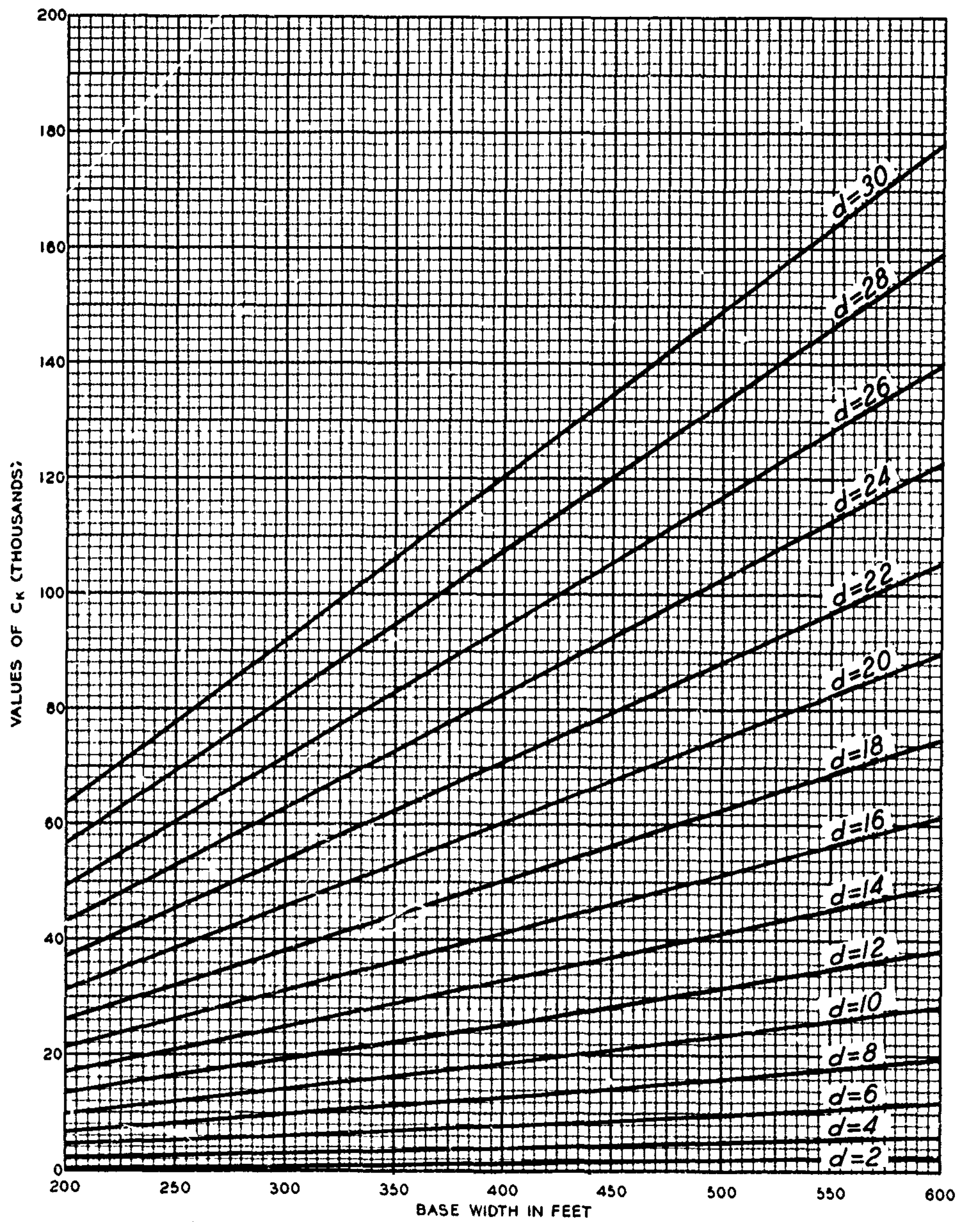
WHERE.
A = AREA
R = HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE 2 TO 1

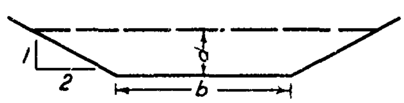
HYDRAULIC DESIGN CHART 610-3

WES 2-54



$C_K = AR^{2/3}$

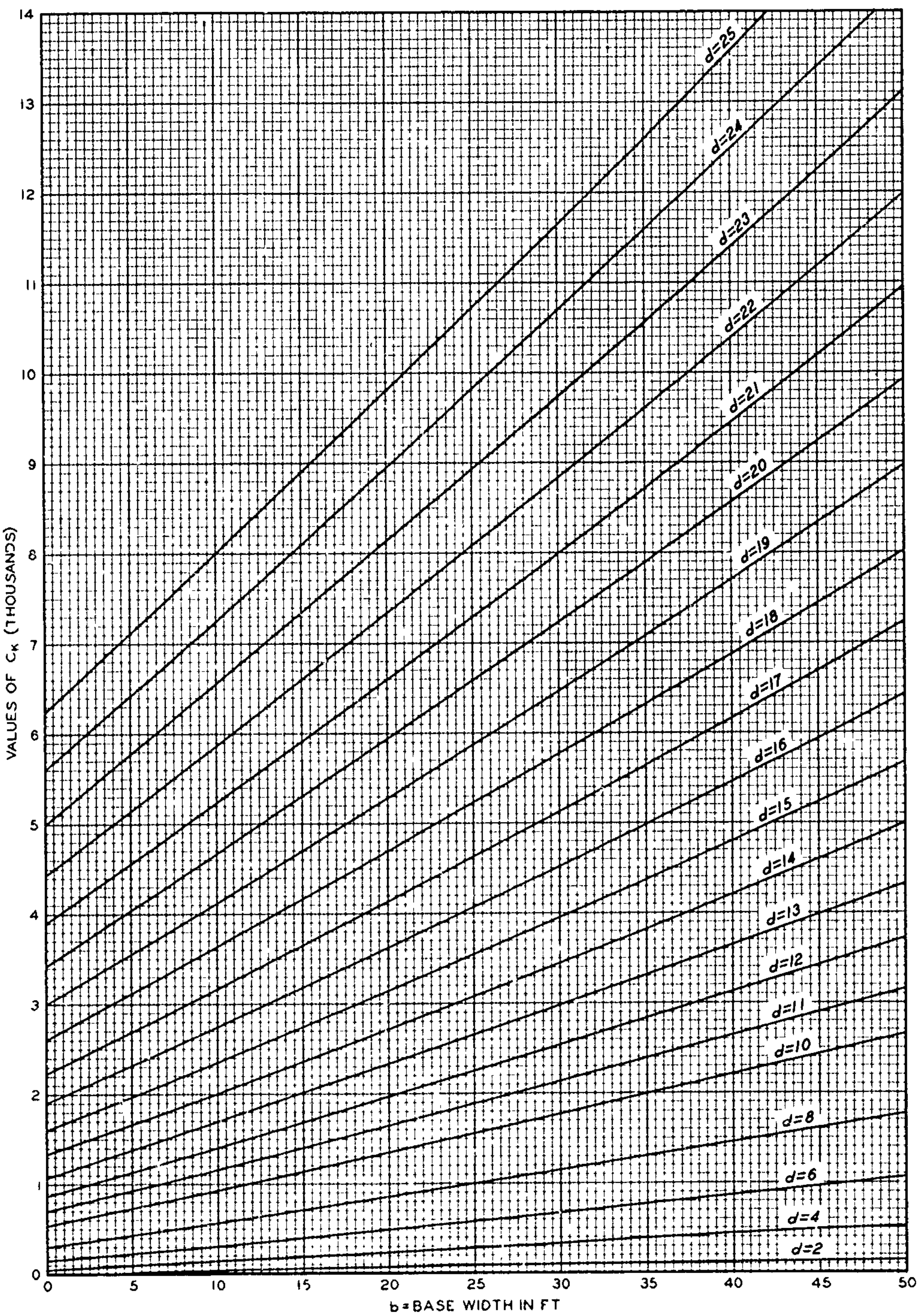
WHERE
A=AREA
R=HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE 2 TO 1

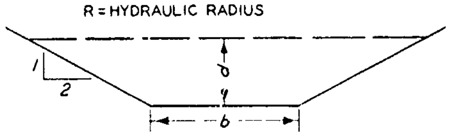
HYDRAULIC DESIGN CHART 610-3/1

WES 9-54



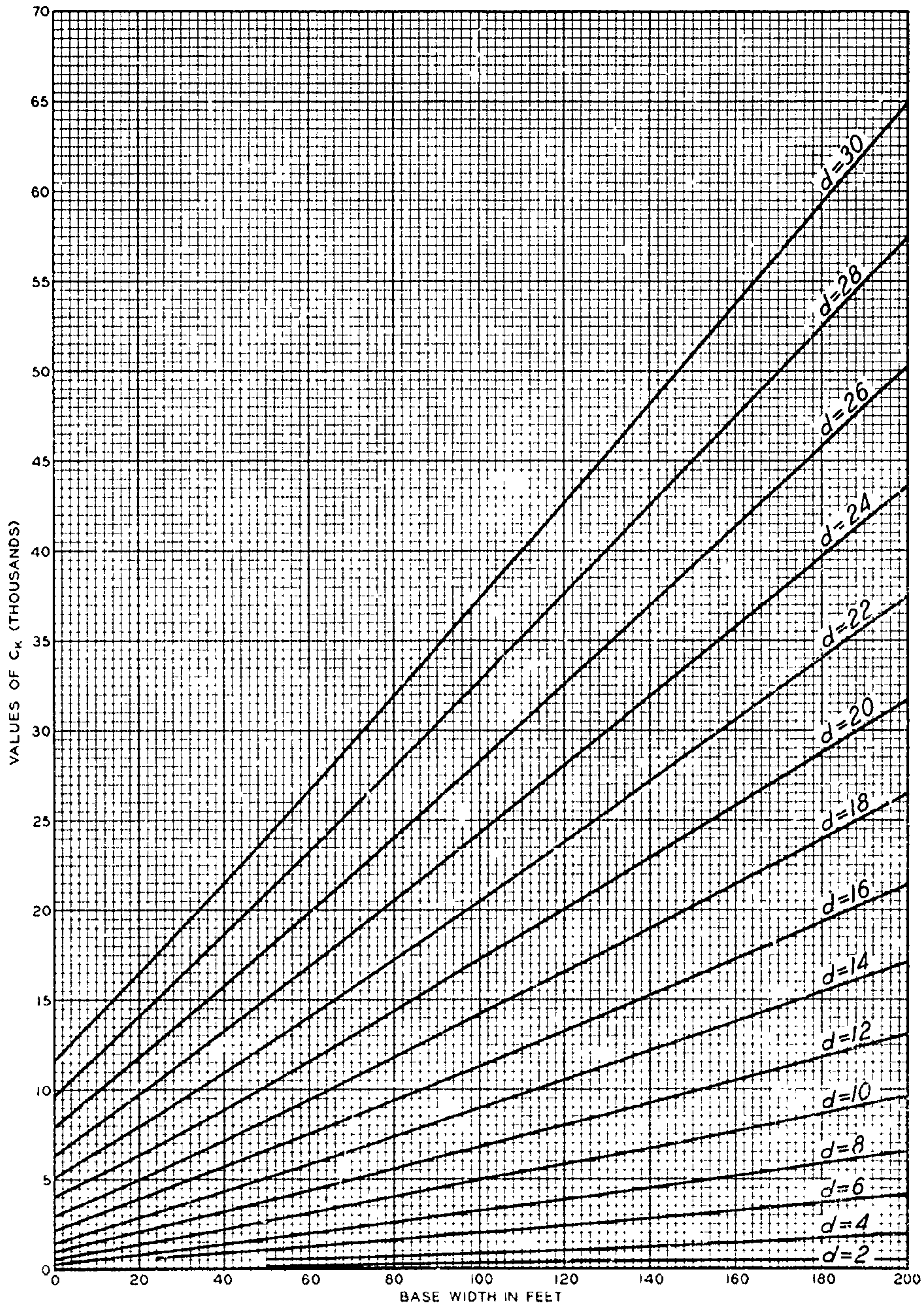
$C_K = AR^{2/3}$

WHERE
A = AREA
R = HYDRAULIC RADIUS



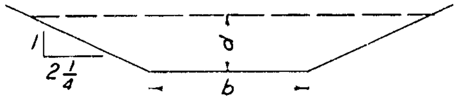
TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE 2 TO 1
BASE WIDTH 0 TO 50 FEET

HYDRAULIC DESIGN CHART 610-3/1-1



$C_K = AR^{2/3}$

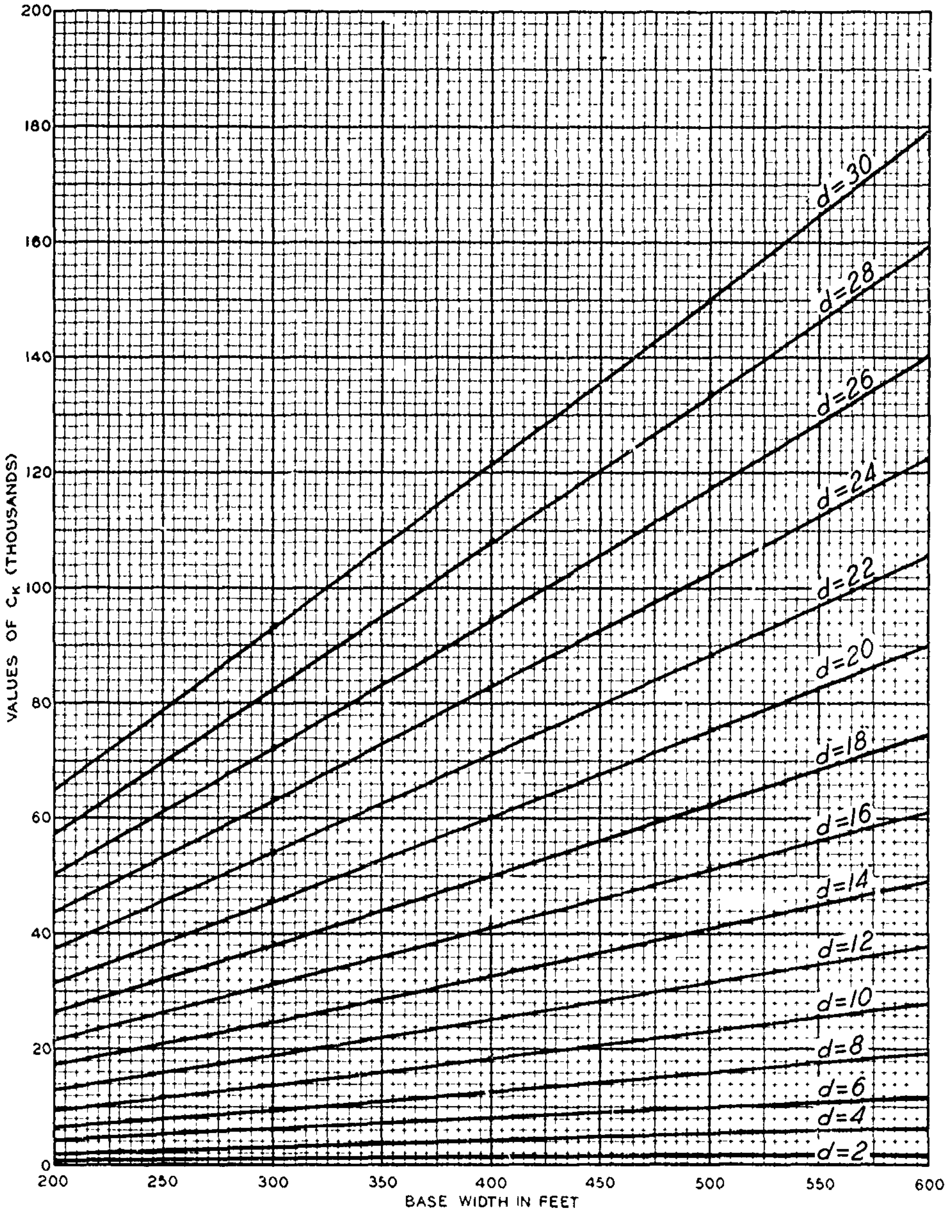
WHERE
A = AREA
R = HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $2\frac{1}{4}$ TO 1

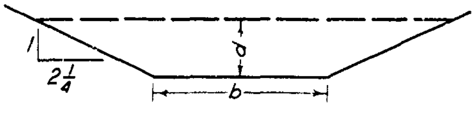
HYDRAULIC DESIGN CHART 610-3/2

WES 9-54



$C_K = AR^{2/3}$

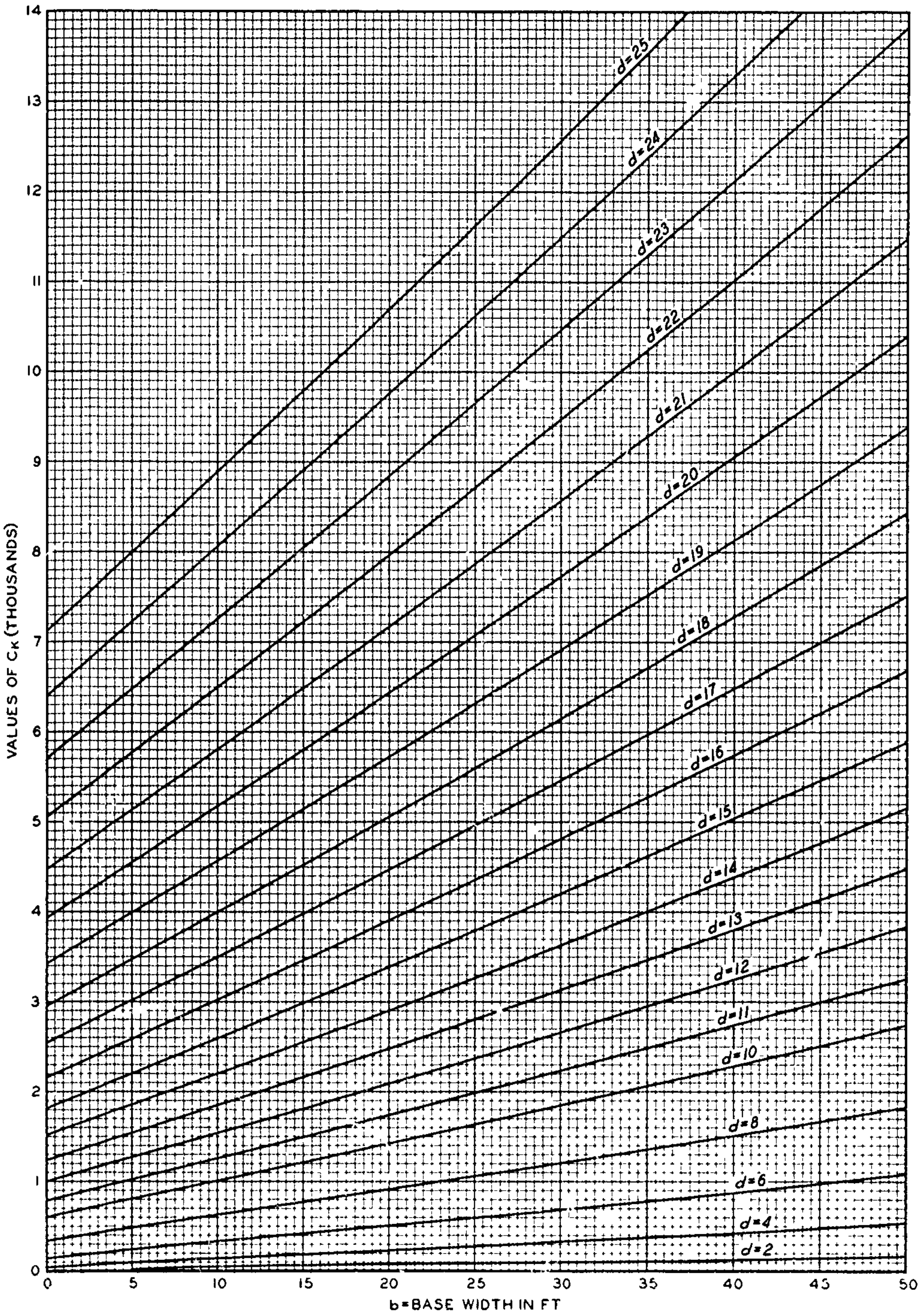
WHERE
A AREA
R HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $2\frac{1}{4}$ TO 1

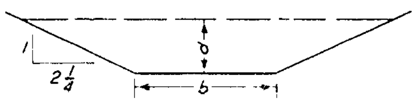
HYDRAULIC DESIGN CHART 610-3/3

WES 9-54



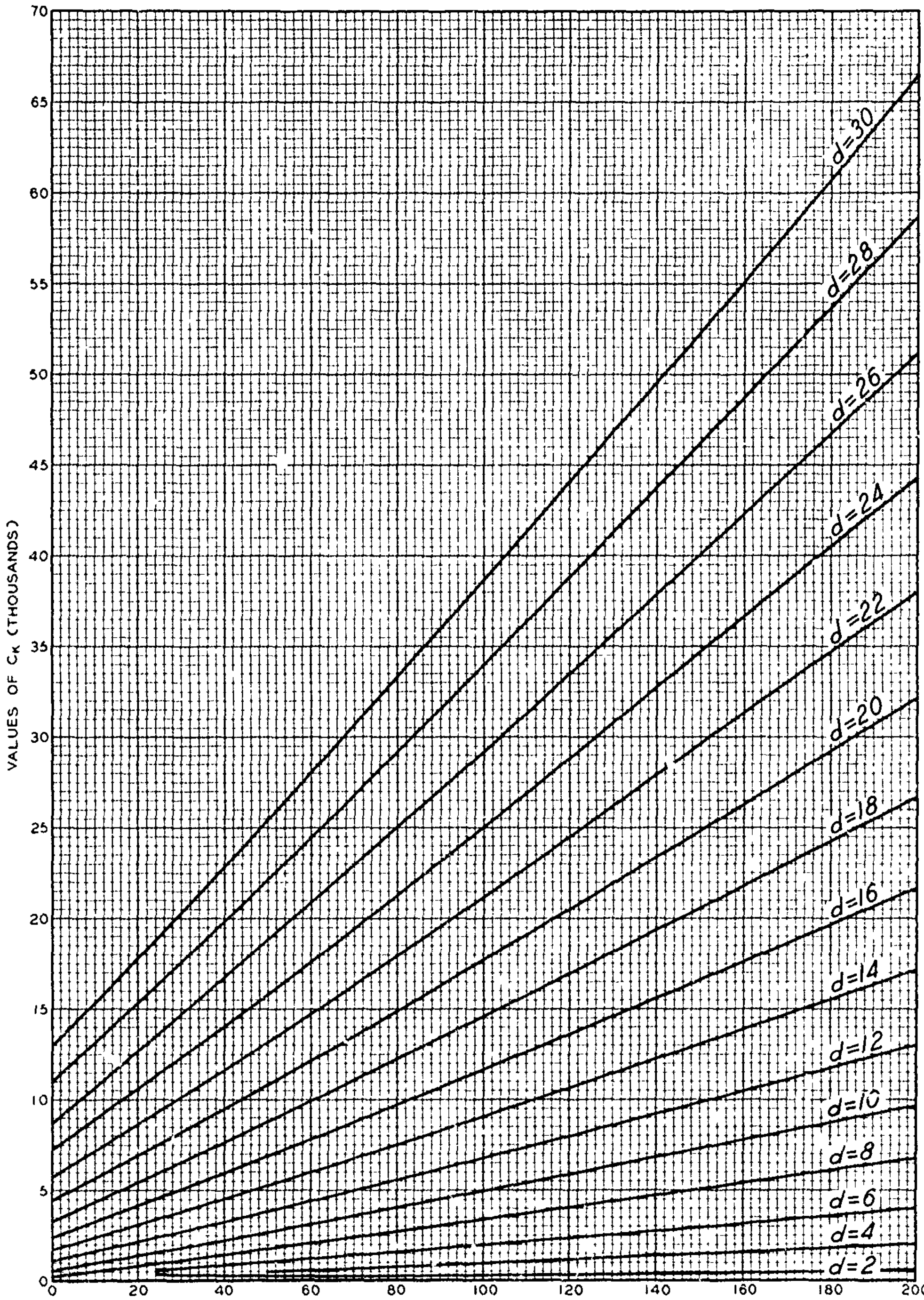
$C_K = AR^{2/3}$

WHERE
A = AREA
R = HYDRAULIC RADIUS



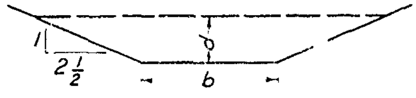
TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $2\frac{1}{4}$ TO 1
BASE WIDTH 0 TO 50 FEET

HYDRAULIC DESIGN CHART 610-3/3-1



$C_K = AR^{2/3}$

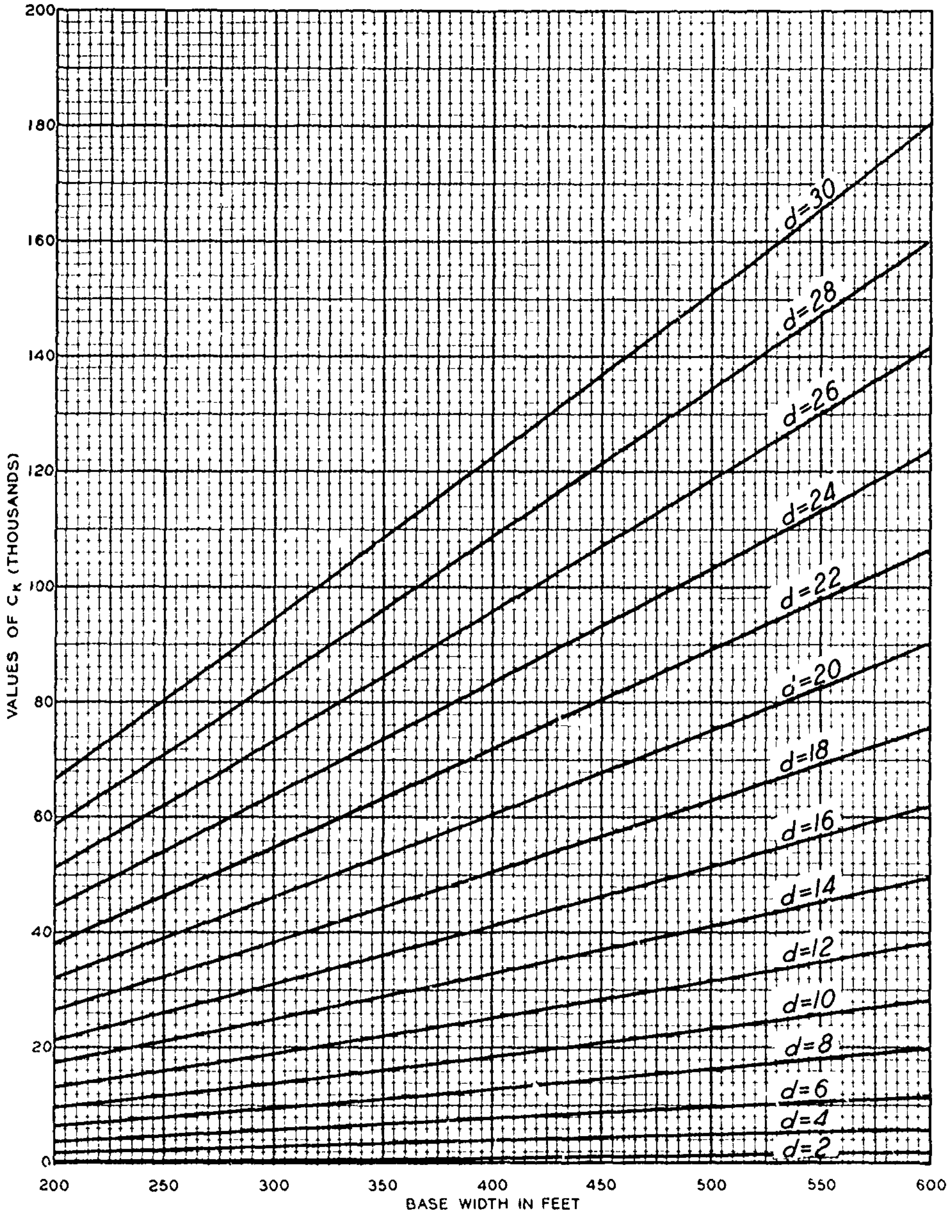
WHERE
A = AREA
R = HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $2\frac{1}{2}$ TO 1

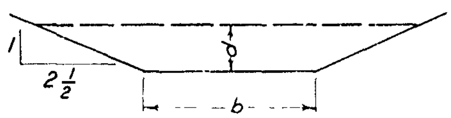
HYDRAULIC DESIGN CHART 610-3/4

WES 9-54



$C_K = AR^{2/3}$

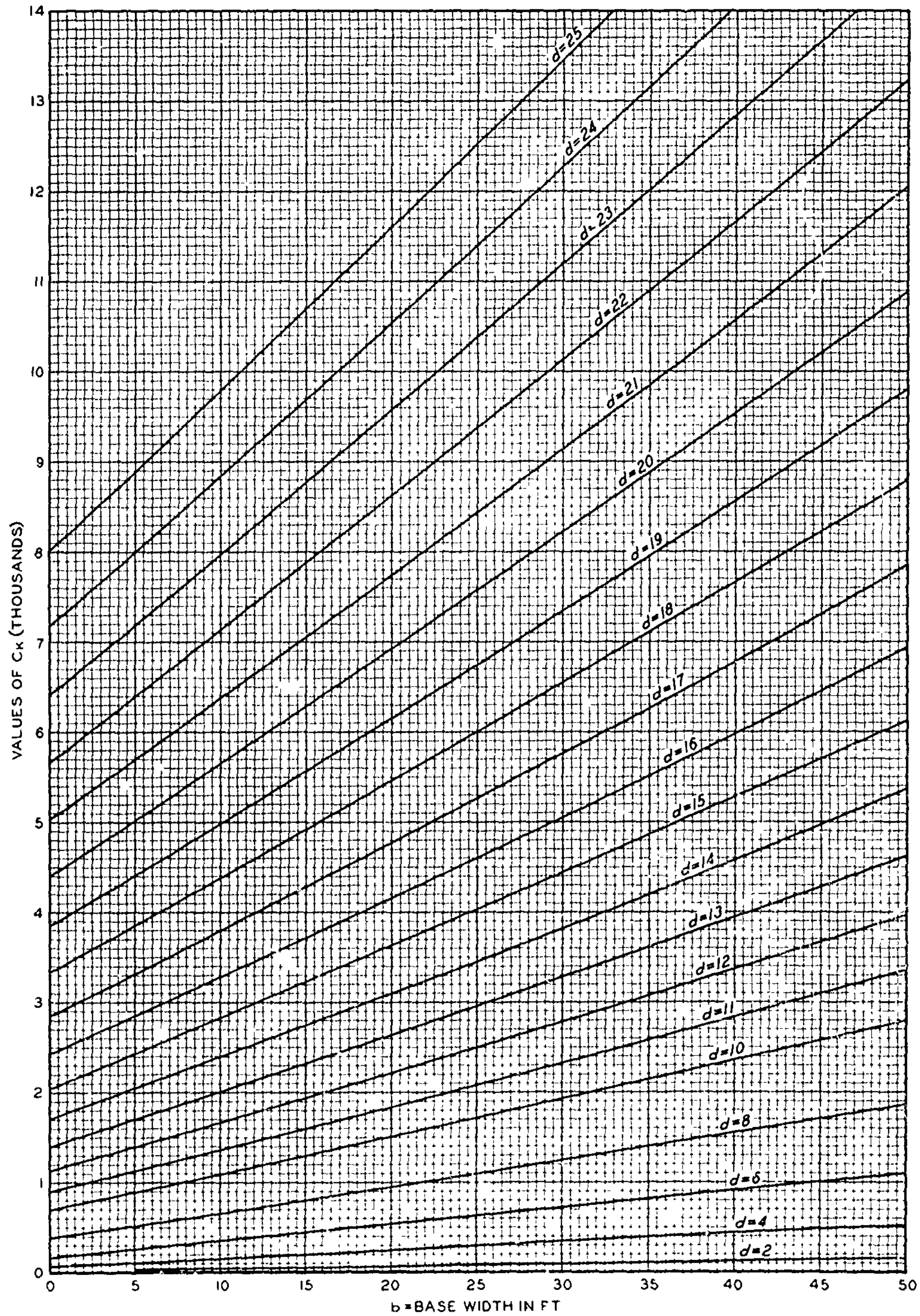
WHERE
A = AREA
R = HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $2\frac{1}{2}$ TO 1

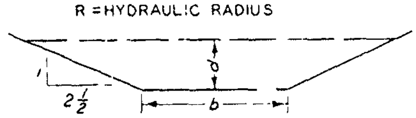
HYDRAULIC DESIGN CHART 610-3/5

WES 9-54



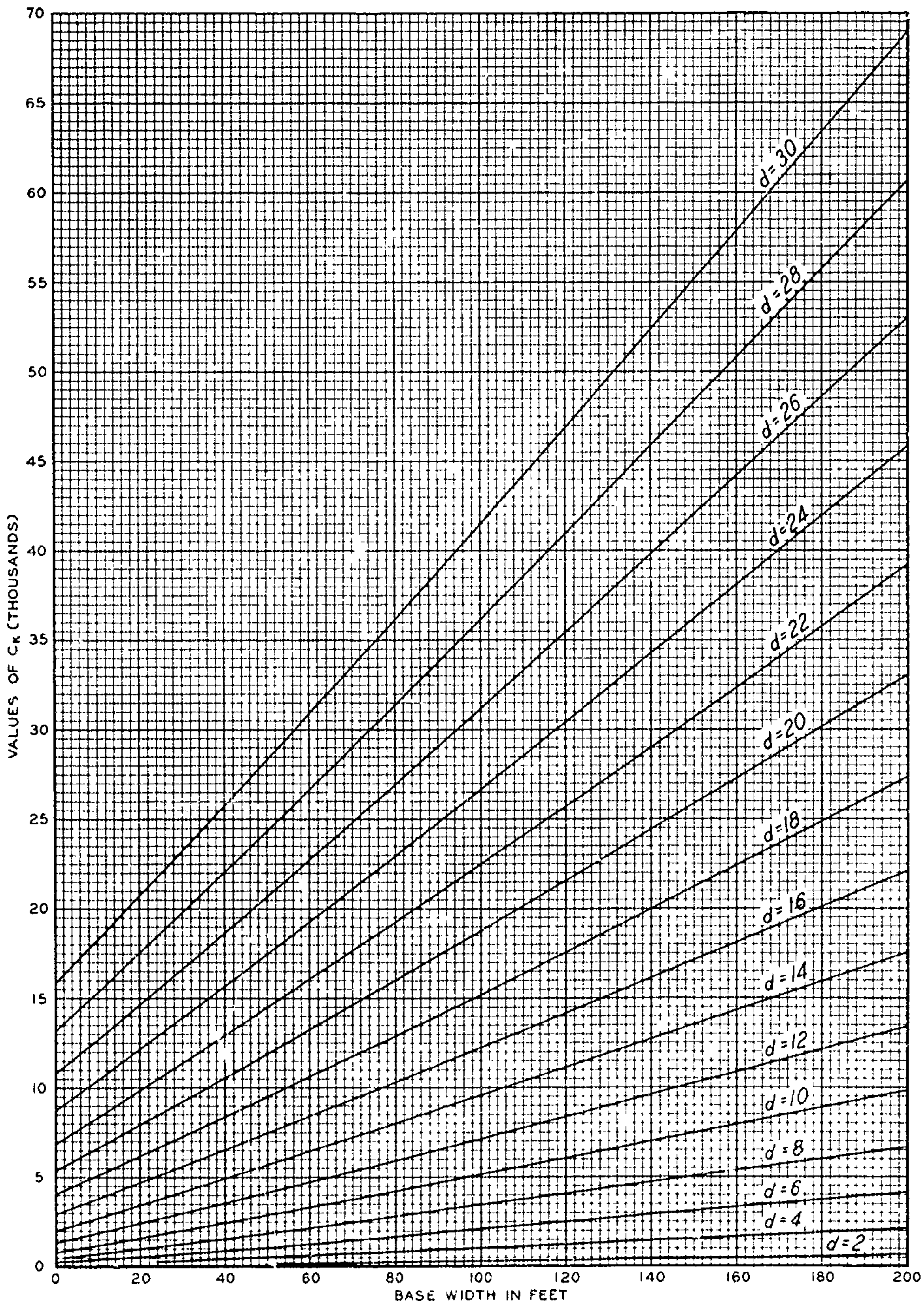
$C_K = AR^{2/3}$

WHERE
A = AREA
R = HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE $2\frac{1}{2}$ TO 1
BASE WIDTH 0 TO 50 FEET

HYDRAULIC DESIGN CHART 610-3/5-1

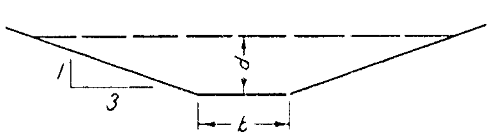


$C_K = AR^{2/3}$

WHERE,

A = AREA

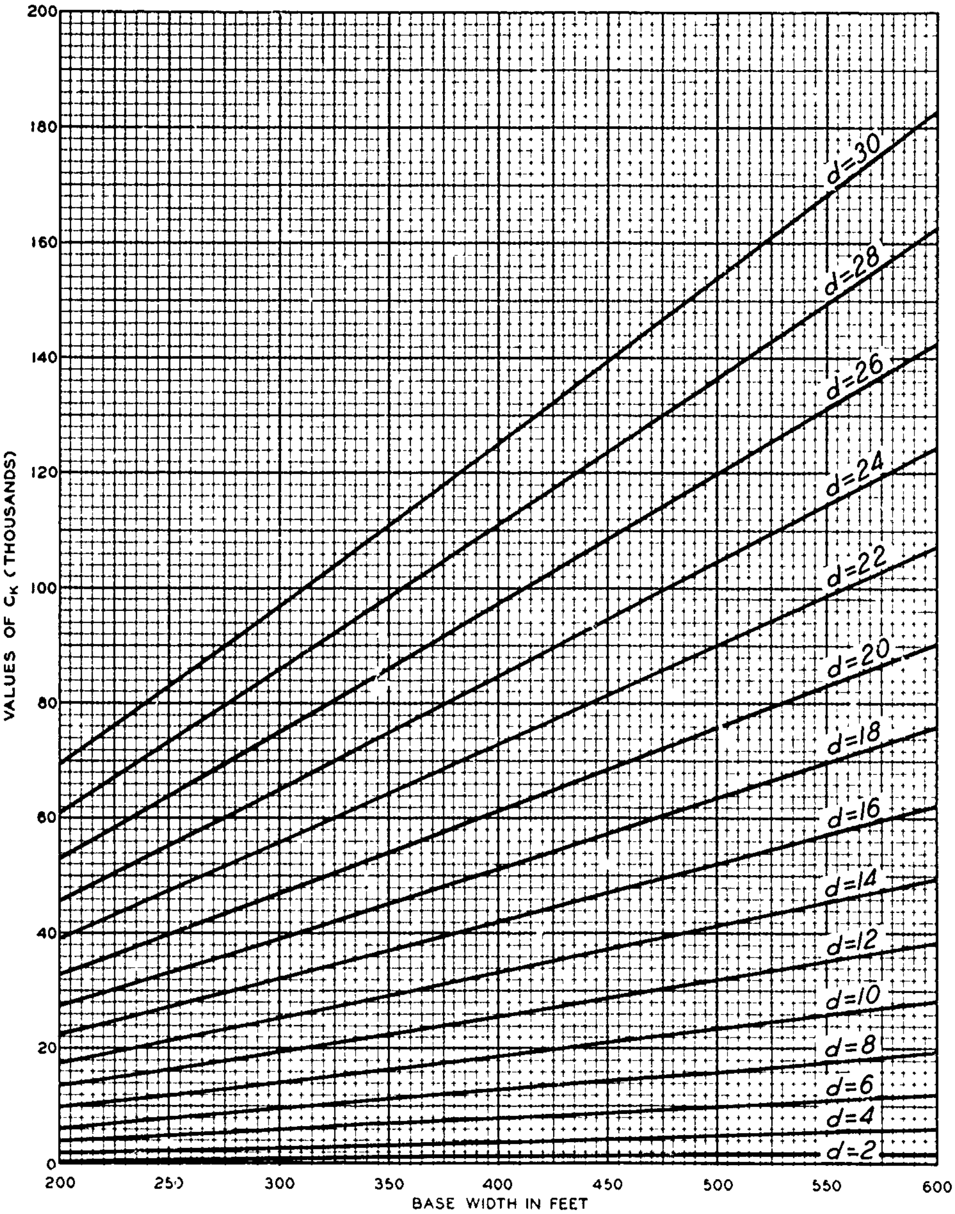
R = HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE 3 TO 1

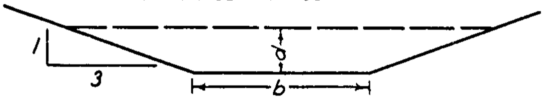
HYDRAULIC DESIGN CHART 610-4

WES 2-54



$C_K = AR^{2/3}$

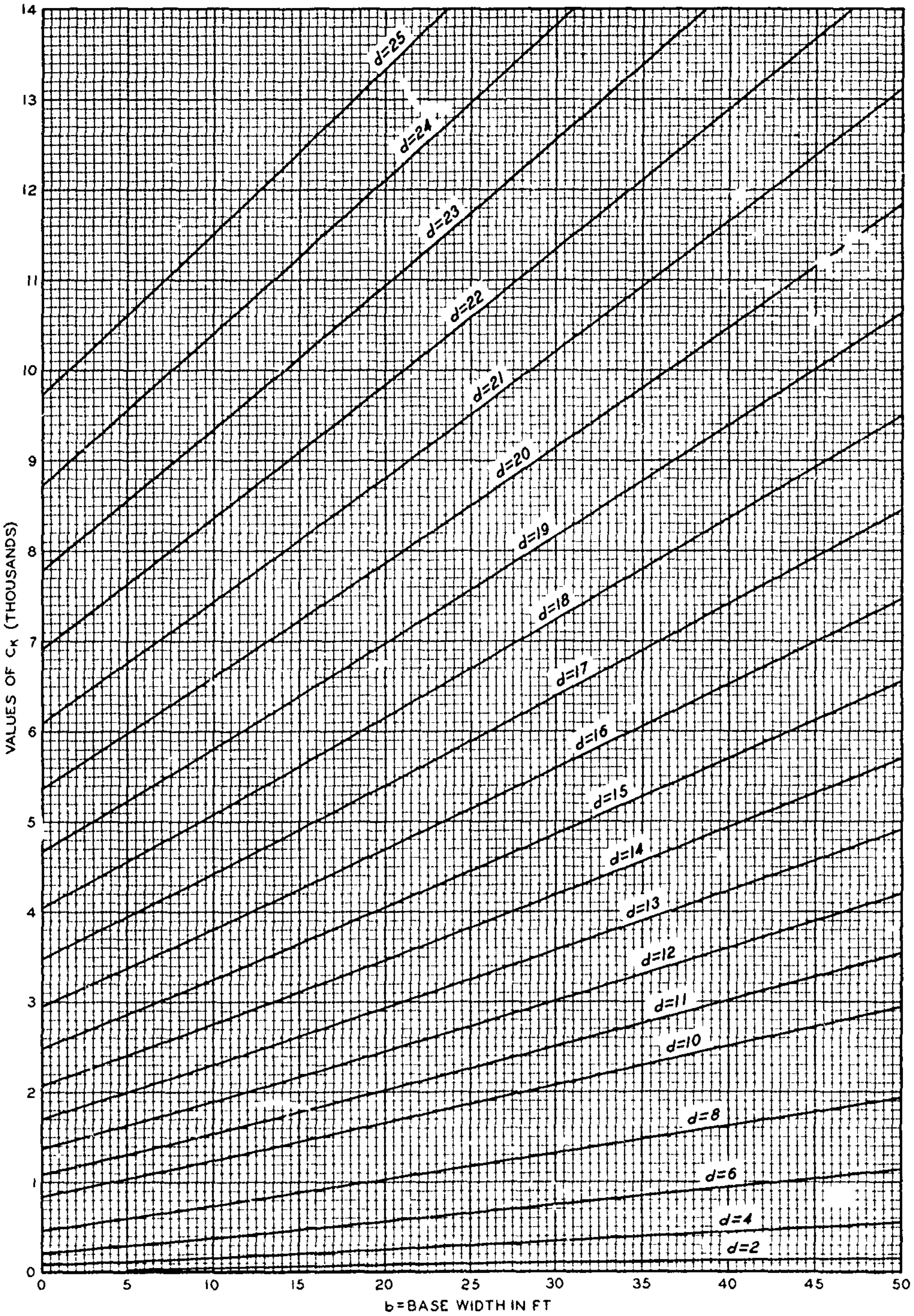
WHERE
A=AREA
R=HYDRAULIC RADIUS



TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE 3 TO 1

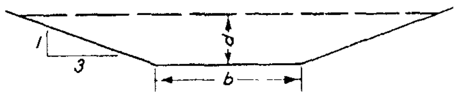
HYDRAULIC DESIGN CHART 610-4/1

WES 9-54



$C_K = AR^{2/3}$

WHERE
A = AREA
R = HYDRAULIC RADIUS

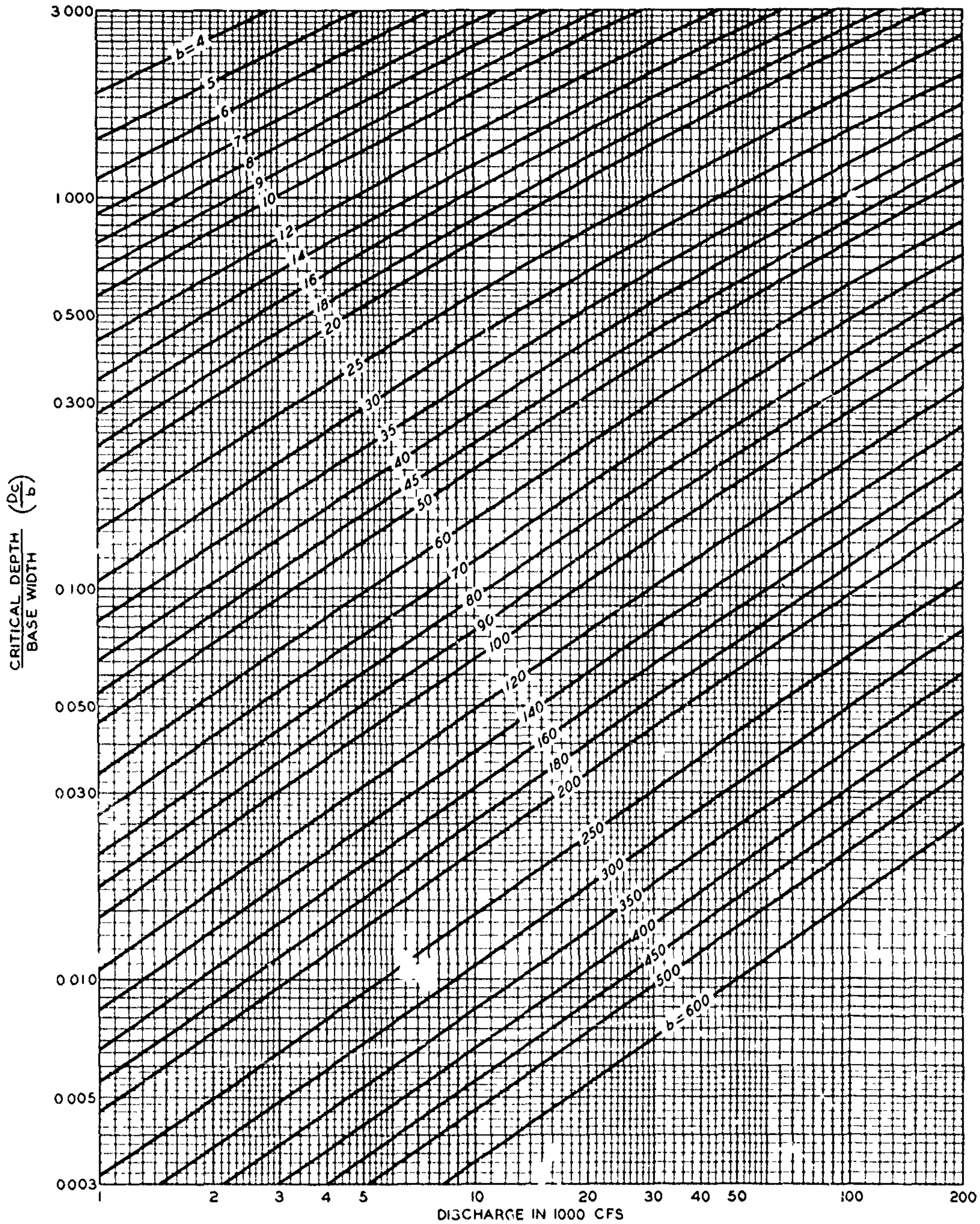


TRAPEZOIDAL CHANNELS
C_K VS BASE WIDTH
SIDE SLOPE 3 TO 1
BASE WIDTH 0 TO 50 FEET

HYDRAULIC DESIGN CHART 610-4/1-1

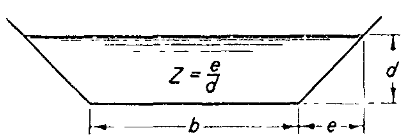
PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 5-59



BASIC FORMULA:

$$Q = D_c^{3/2}\sqrt{\frac{(b + ZD_c)^3}{b + 2ZD_c} \times g}$$

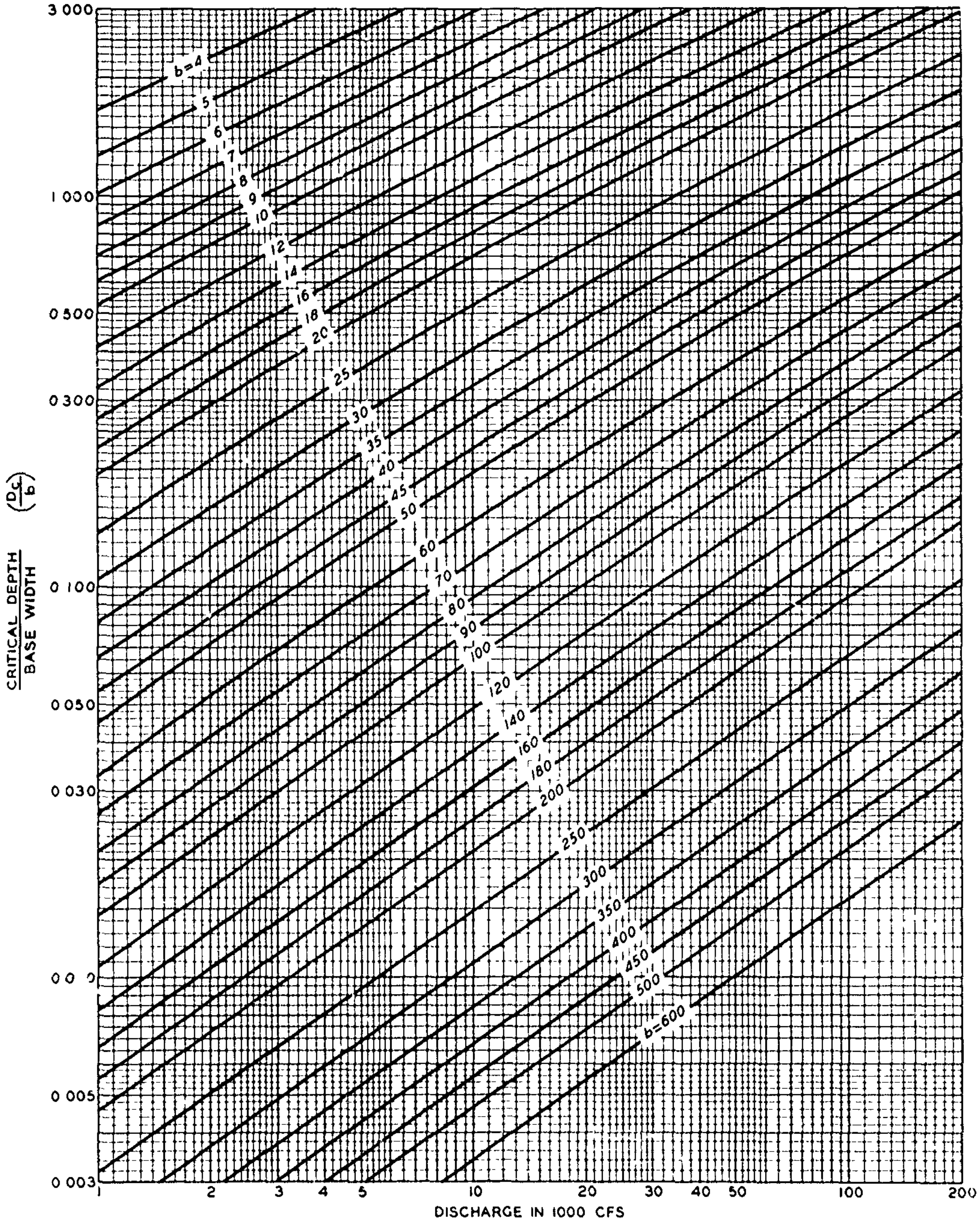


TRAPEZOIDAL CHANNELS CRITICAL DEPTH CURVES

SIDE SLOPE 1 TO 1

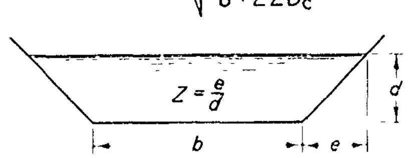
HYDRAULIC DESIGN CHART 610-5

WES 2-54



BASIC FORMULA:

$$Q = D_c^{3/2}\sqrt{\frac{(b + ZD_c)^3}{b + 2ZD_c} \times g}$$

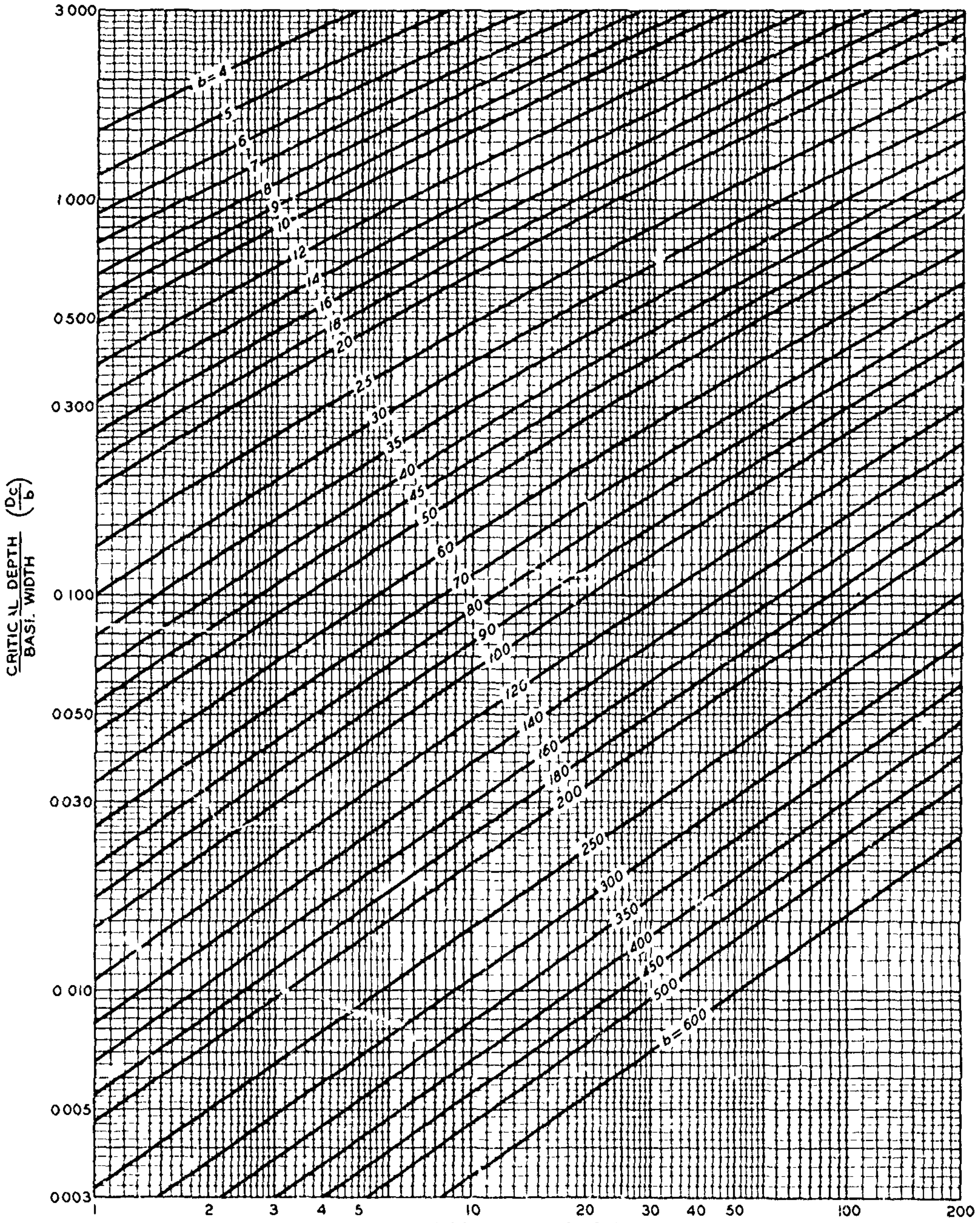


TRAPEZOIDAL CHANNELS CRITICAL DEPTH CURVES

SIDE SLOPE $1\frac{1}{2}$ TO 1

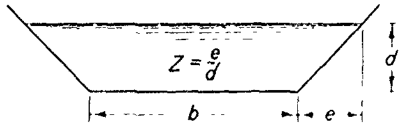
HYDRAULIC DESIGN CHART 610-5/1

WES 9-54



BASIC FORMULA

$$Q = D_c^{3/2}\sqrt{\frac{(b+ZD_c)^3}{b+2ZD_c} \times g}$$

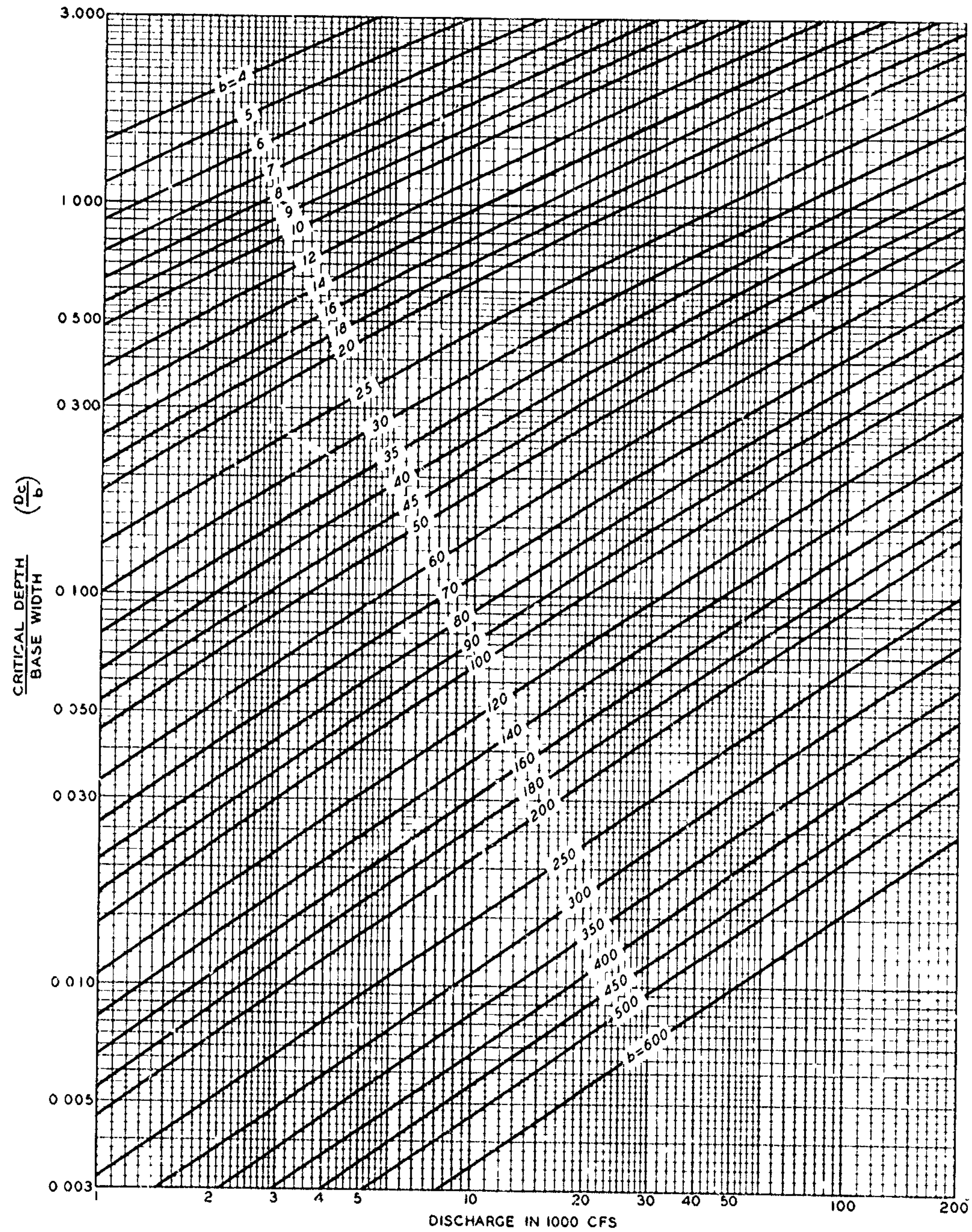


TRAPEZOIDAL CHANNELS CRITICAL DEPTH CURVES

SIDE SLOPE 2 TO 1

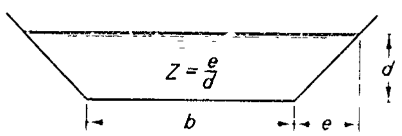
HYDRAULIC DESIGN CHART 610-6

WES 2-54



BASIC FORMULA·

$$) \ {}_c^{3/2}\sqrt{\frac{(b+ZD_c)^3}{b+2ZD_c}\times g}$$

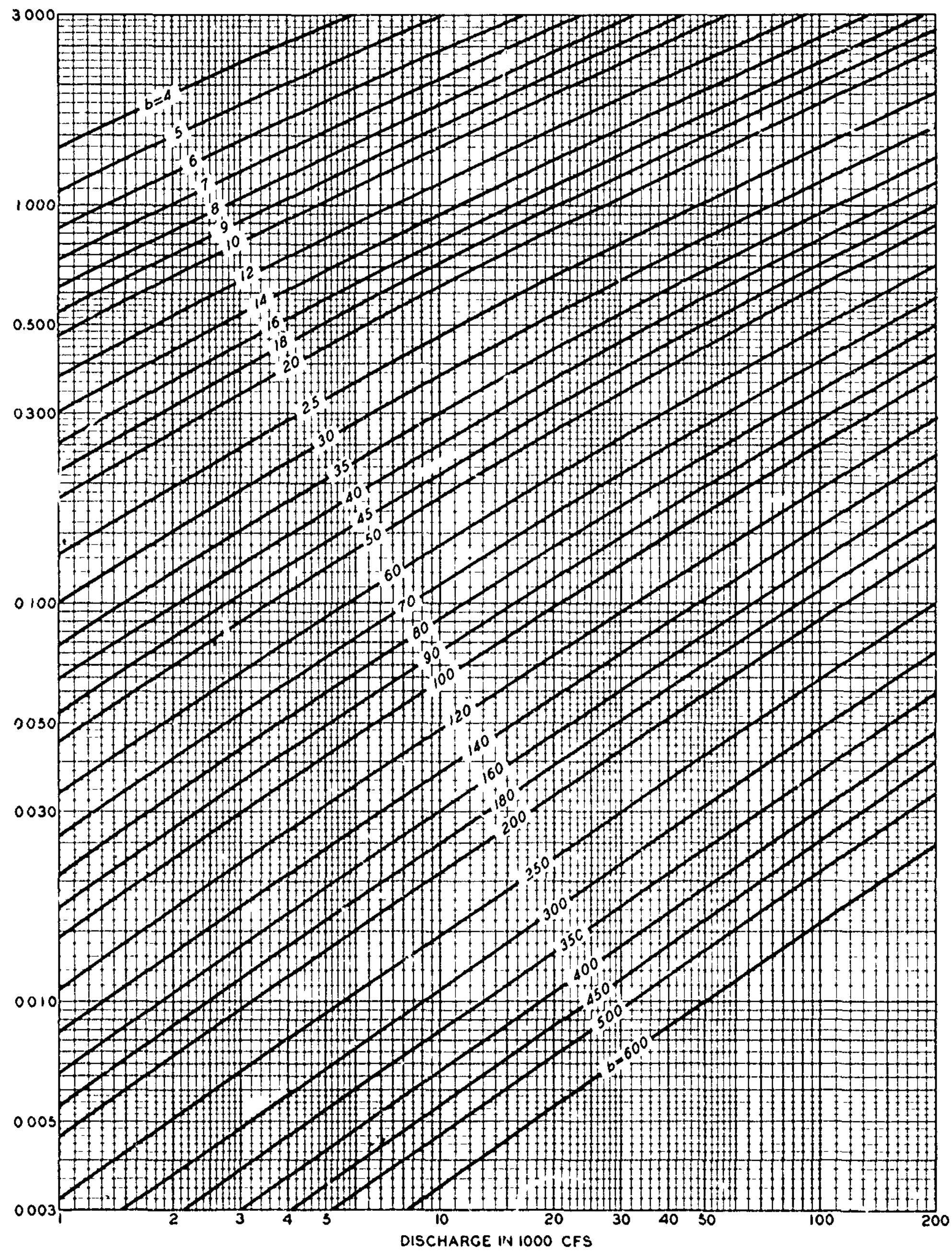


TRAPEZOIDAL CHANNELS CRITICAL DEPTH CURVES

SIDE SLOPE $2\frac{1}{4}$ TO 1

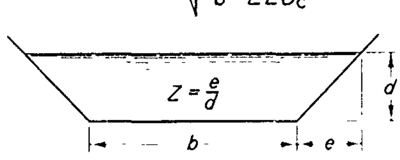
HYDRAULIC DESIGN CHART 610-6/1

WES 9-54



BASIC FORMULA:

$$Q = D_c^{3/2}\sqrt{\frac{(b+ZD_c)^3}{b+2ZD_c} \times g}$$

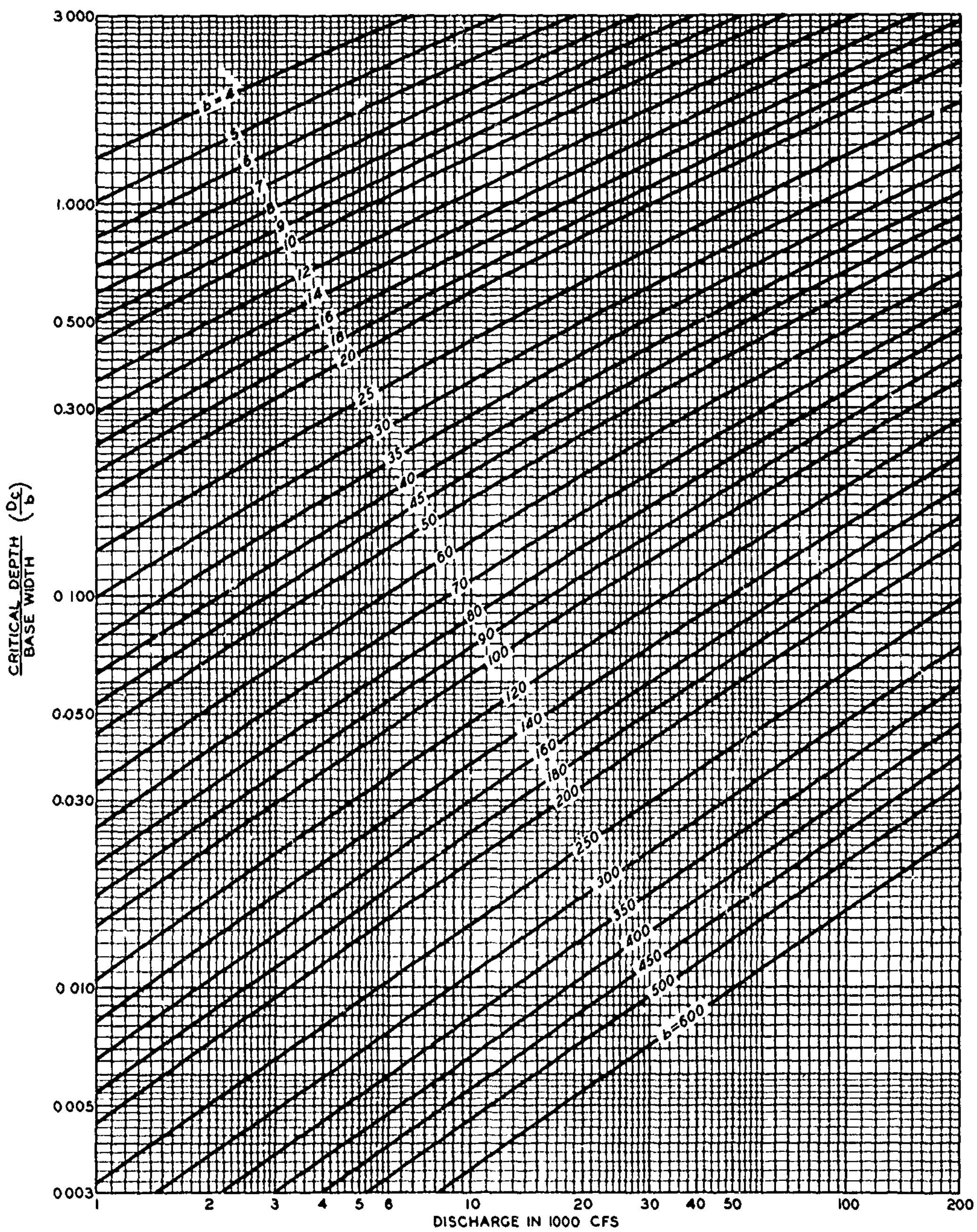


TRAPEZOIDAL CHANNELS CRITICAL DEPTH CURVES

SIDE SLOPE $2\frac{1}{2}$ TO 1

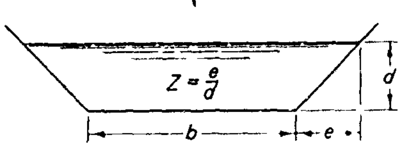
HYDRAULIC DESIGN CHART 610-6/2

WES 9-54



BASIC FORMULA:

$$Q = D_c^{3/2}\sqrt{\frac{(b + ZD_c)^3}{b + 2ZD_c} \times g}$$



TRAPEZOIDAL CHANNELS CRITICAL DEPTH CURVES

SIDE SLOPE 3 TO 1

HYDRAULIC DESIGN CHART 610-7

WES 2-54

HYDRAULIC DESIGN CRITERIA

SHEETS 610-8 TO 610-9/1-1

OPEN CHANNEL FLOW

RECTANGULAR SECTIONS

1. Hydraulic Design Charts 610-8 to 610-9/1-1 are aids for reducing the computation effort in the design of rectangular channels. These charts are useful also in the backwater computations presented on Chart 010-2.

2. Basic Equations. Chart 610-8 shows plots of normal depth (y_o) with respect to discharge per foot of width (q) for wide rectangular sections where the side wall effect may be neglected. Normal depth curves are shown for Manning's n of 0.011 and 0.013 and for slopes of 0.01 to 0.50. The roughness and slopes values are those commonly used in the design of spillway chutes. The curves are computed from a variation of the Manning formula for open channel flow.

$$q = cy_o^{5/3}$$

where

$$c = \frac{1.486\ S^{1/2}}{n}$$

Critical depth (y_c) with respect to q is also plotted on this chart. Critical depth in rectangular channels is a function of unit discharge only

$$y_c = \sqrt[3]{\frac{q^2}{g}}$$

3. Charts 610-9 through 610-9/1-1 in conjunction with Charts 610-1 and -1/1 can be used to determine normal depths (y_o) for any rectangular channel. These charts are similar to Charts 610-2 to 610-4/1-1 and were developed in the manner described in paragraph 2 of Sheets 610-1 to 610-7.

4. Application. Preliminary design of rectangular channels for uniform subcritical or supercritical flows is readily determined by use of the charts in the following manner:

   a. Two-dimensional flow. For wide channels, y_o and y_c can be obtained directly from Chart 610-8 for given values of n, S, and q.

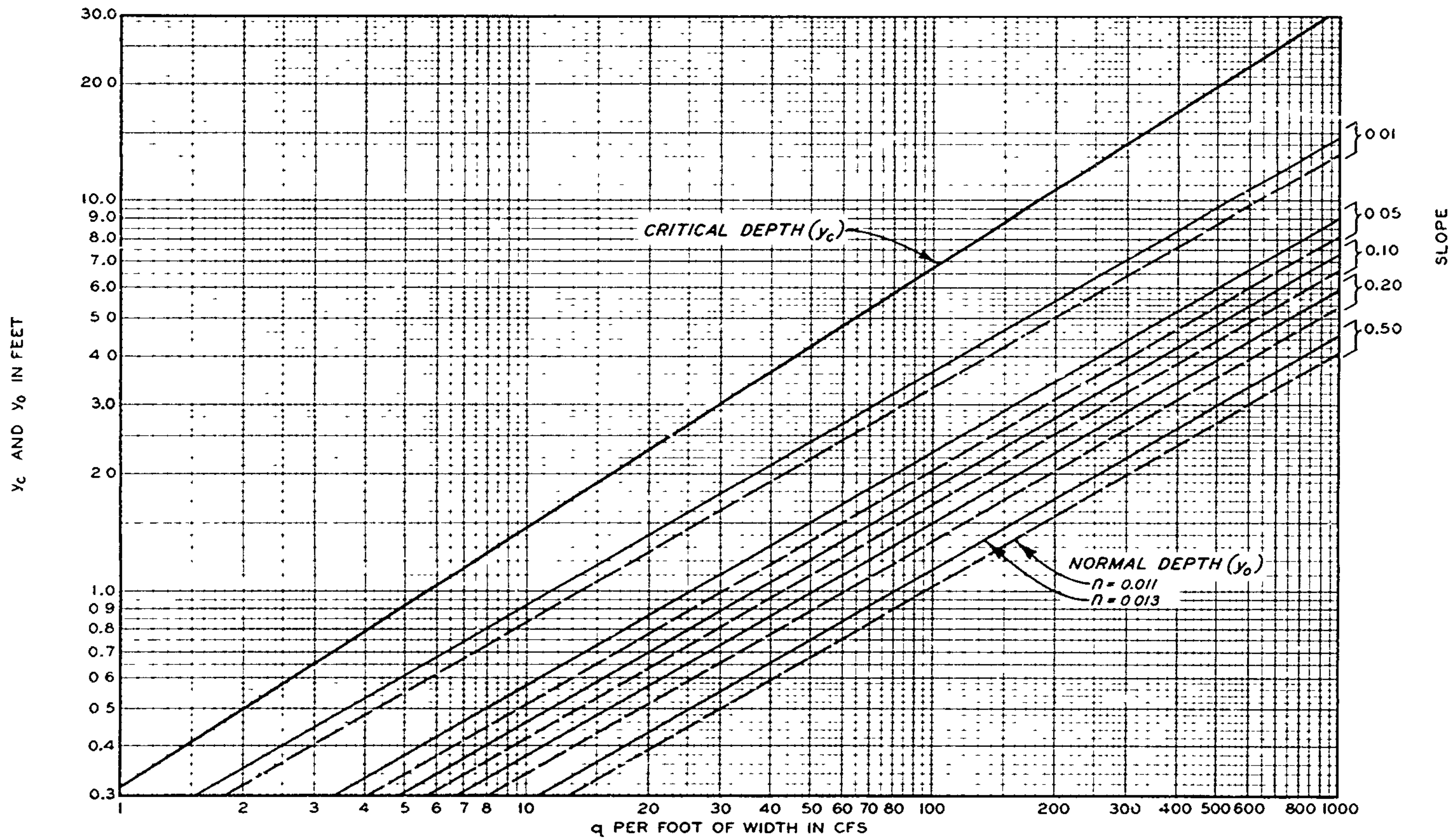
b. Three-dimensional flow. For all channels, Charts 610-9 through 610-9/1-1 can be used in the manner described in paragraphs 3a, b, and c, Sheets 610-1 to 610-7. Critical depth can be obtained from Chart 610-8.

c. Normal depth for three-dimensional flow can also be computed from Chart 610-8 by use of the following table:

b/d_2	d_3/d_2
2	1.38
5	1.17
10	1.07
15	1.05
25	1.03

where

b = channel width in ft
d_2 = two-dimensional flow depth in ft
d_3 = three-dimensional flow depth in ft.



BASIC EQUATIONS

$q = C y_o^{5/3}, \quad y_c = \sqrt[3]{\frac{q^2}{g}}$

WHERE: q = DISCHARGE PER FOOT OF WIDTH

$C = \frac{1.486\, S^{1/2}}{n}$

y_o = NORMAL DEPTH IN FEET

y_c = CRITICAL DEPTH IN FEET

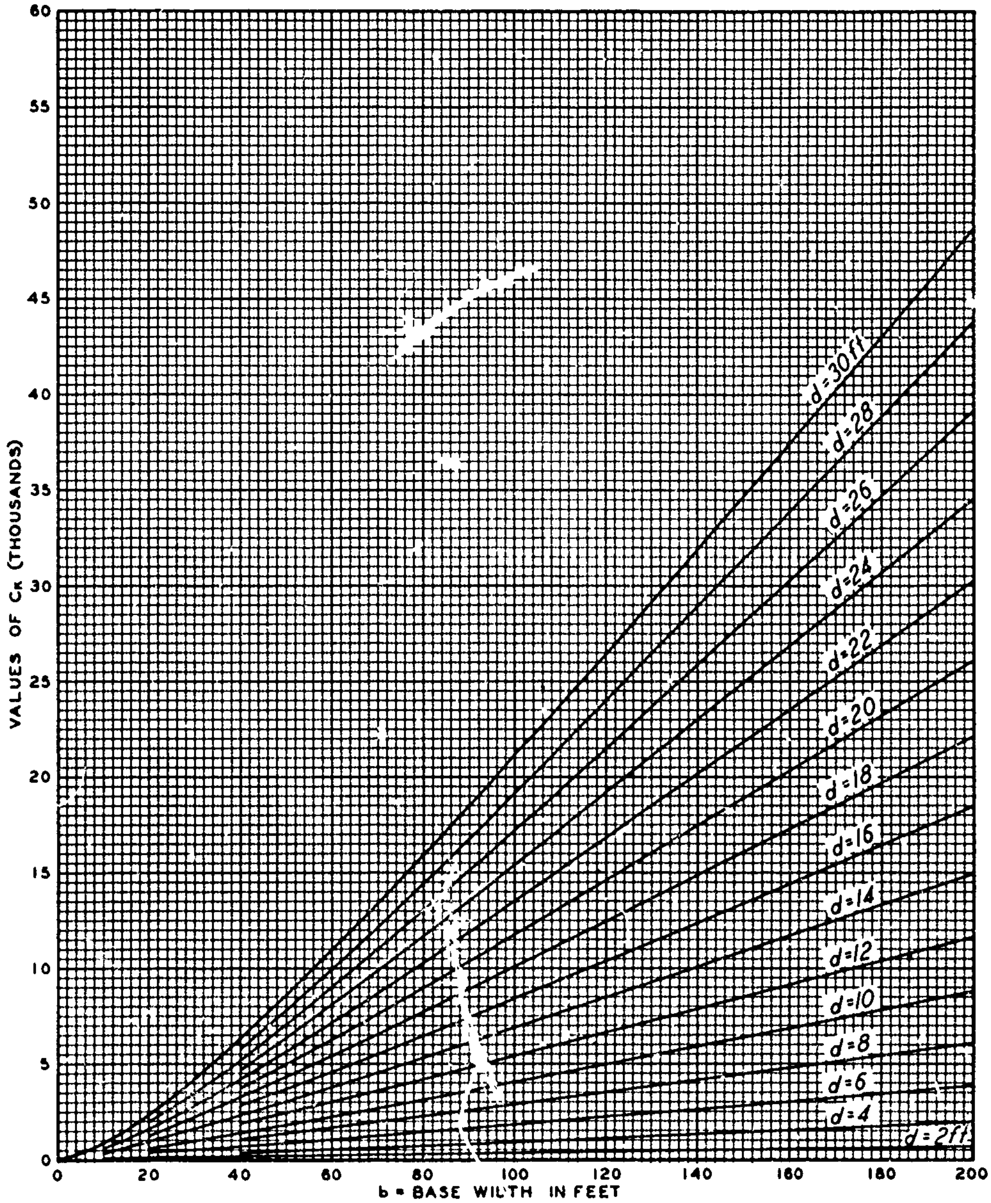
g = GRAVITY

OPEN CHANNEL FLOW

NORMAL AND CRITICAL DEPTHS WIDE RECTANGULAR SECTIONS

HYDRAULIC DESIGN CHART 610-8

WES 3-56



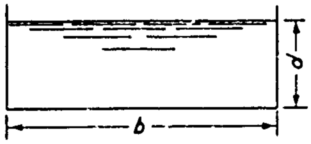
BASIC EQUATION

$C_K = AR^{2/3}$

WHERE

A = AREA

R = HYDRAULIC RADIUS



OPEN CHANNEL FLOW

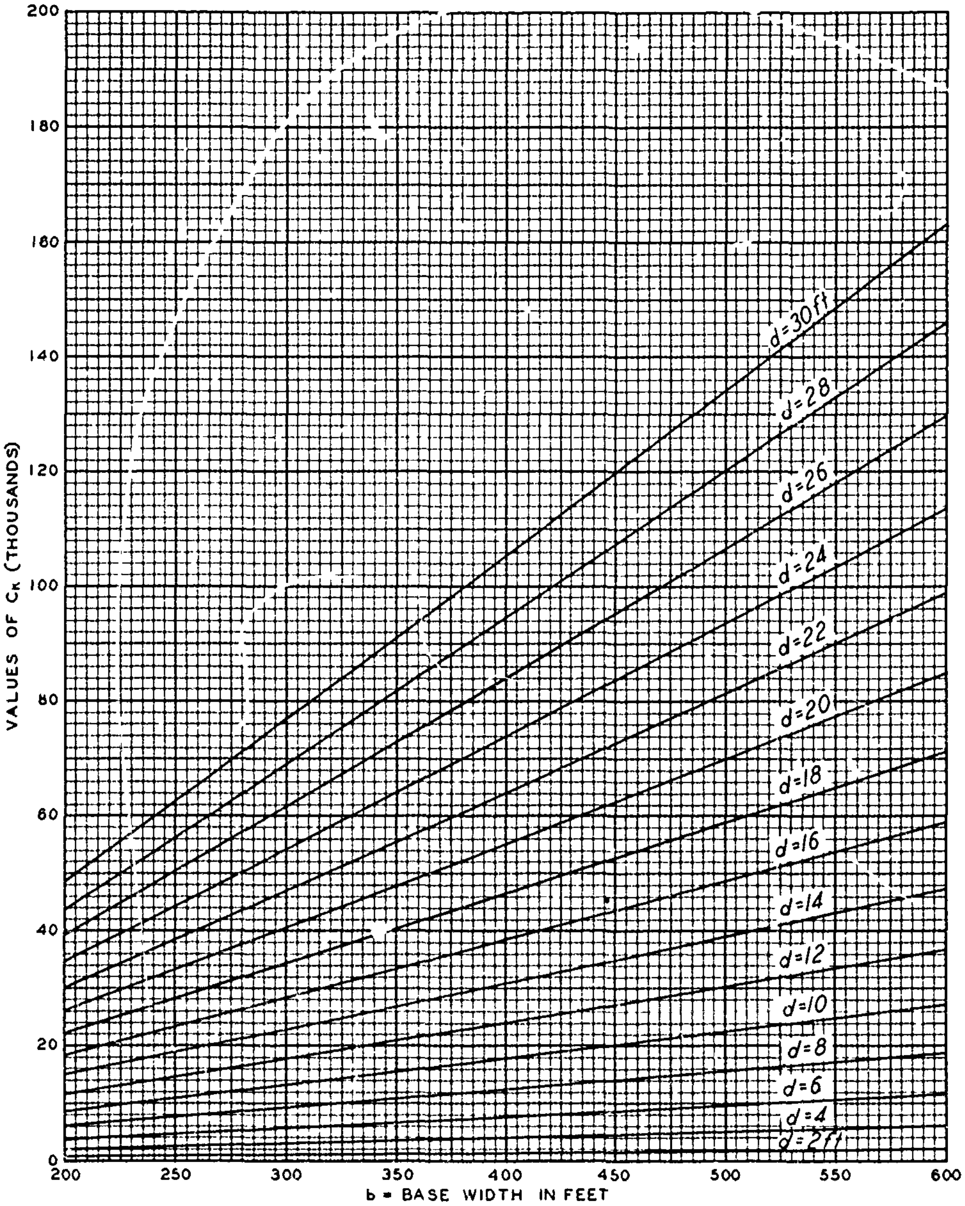
C_K VS BASE WIDTH

RECTANGULAR SECTIONS

BASE WIDTHS OF 0 TO 200 FT

HYDRAULIC DESIGN CHART 610-9

WES 3-56



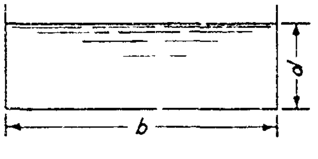
BASIC EQUATION

$C_K = AR^{2/3}$

WHERE

A = AREA

R = HYDRAULIC RADIUS



OPEN CHANNEL FLOW

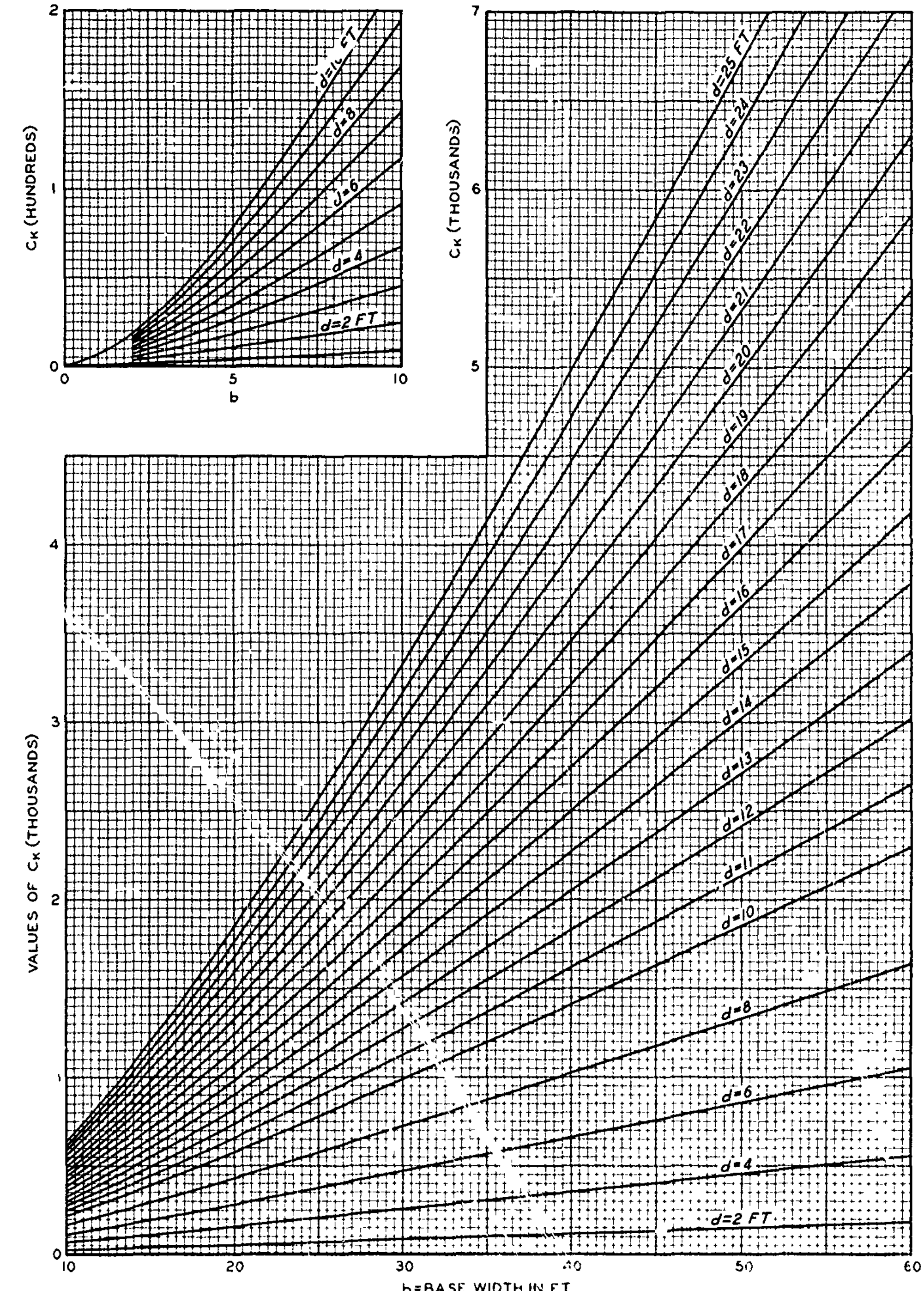
C_K VS BASE WIDTH

RECTANGULAR SECTIONS

BASE WIDTHS OF 200 TO 600 FT

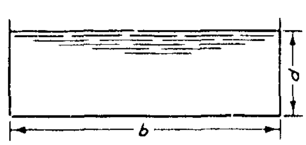
HYDRAULIC DESIGN CHART 610-9/1

WES 3-56



$C_K = AR^{2/3}$

WHERE
A = AREA
R = HYDRAULIC RADIUS



OPEN CHANNEL FLOW
C_K VS BASE WIDTH
RECTANGULAR SECTIONS
BASE WIDTH 0 TO 60 FEET

HYDRAULIC DESIGN CHART 610-9/1-1

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

WES 5-59

HYDRAULIC DESIGN CRITERIA

SHEETS 623 TO 624-1

SUBCRITICAL OPEN CHANNEL FLOW

DROP STRUCTURES

1. Purpose. A channel invert slope can vary from a maximum defined by a line connecting the crests of two drop structures to a minimum fixed by the elevation of the end sill of the upstream structure, the elevation of the crest of the downstream structure, and the distance between the two structures. The minimum slope should be that which results in stable channel conditions.

2. Hydraulic Design Charts (HDC's) 623 to 624-1 present design criteria for drop structures in subcritical flow used to prevent channel degradation. The criteria shown in HDC 623 are recommended for drops where the unit discharge is large relative to the drop height. The design criteria shown in HDC 624 and 624/1 are recommended for drop structures where both the unit discharge and drop height are large and where optimum energy dissipation is required to reduce downstream erosion. In most cases economy of construction is the deciding factor.

3. Background. The accepted relation between the height of drop h (difference in elevation between the crest and the end sill of the drop structure), critical depth d_c at the drop, and the required stilling basin length L_B is attributed to Etcheverry[1] and defined by the equation

$$L_B = C_L \sqrt{hd_c} \tag{1}$$

where C_L is an empirical apron length coefficient. Studies by Morris and Johnson[2] resulted in design of the CIT (California Institute of Technology) structure restricted to h/d_c ratios greater than 1.0. Subsequent studies by Vanoni and Pollak[3] included ratios as low as 0.3. While initial research efforts were directed toward erosion control in gullies, subsequent application has been mostly in alluvial streams.

4. Donnelly and Blaisdell[4] investigated drop structures having h/d_c ratios from 1 to 15 and developed the SAF drop structure for primary use in the control of erosion in gullies. The major difference in CIT and SAF structures is the difference in tailwater depths, i.e. shallow and deep, respectively.

5. CIT-Type Drop Structures. Extensive WES tests[5] on the CIT-type structure resulted in the design criteria given in HDC 623. The Vanoni and Pollak results appear to correlate well with the WES tests. WES tests showed that optimum structure performance is obtained if the

structure is designed to have a tailwater-critical depth ratio between 1.25 and 1.67. This results in a strong ground roller, a confined, strong and stable surface roller, and a depressed secondary roller downstream. Curved, upstream abutment walls are recommended for narrow channels to help prevent concentration of the flow. For wide channels with flow width ≥ 20 times the depth, rectangular abutments are satisfactory. Stilling basin training walls should be sufficiently high to prevent the tailwater returning over the walls into the stilling basin. Wing walls at the end of the basin are not recommended. The channel edge should be recessed as indicated in HDC 623.

6. SAF-Type Drop Structures. The SAF-type drop structure[4,6] (HDC's 624 and 624-1) is recommended for designs having large unit discharges and drop heights. The basic layout is shown in HDC 624. The primary controlling parameter in this design is the location at which the upper nappe of the falling jet impinges on the stilling basin floor. This is a function of the total fall of the jet and the depth of the tailwater. Dimensionless curves for determining the impact location of the upper nappe on the basin floor are shown in HDC 624-1.

7. The dimensions of the stilling basin are computed from the following equations.

$$L_B = X_a + X_b + X_c \tag{2}$$

where L_B equals basin length. HDC 624 graphically defines the distance X_a , X_b , and X_c . Numerical values of X_b and X_c are obtained from the following equations:

$$X_b = 0.8d_c \tag{3}$$

$$X_c = 1.75d_c \tag{4}$$

Substituting equations 3 and 4 into equation 1 results in

$$L_B = X_a + 2.55d_c \tag{5}$$

with d_c as defined in paragraph 3 and as shown in HDC's 623 and 624. Laboratory tests[4] have resulted in the following recommendations for baffle pier and end sill heights.

$$\text{Baffle pier height} = 0.8d_c \tag{6}$$

$$\text{End sill height } h' = 0.4d_c \tag{7}$$

These tests also showed that optimum basin performance occurs when the baffle pier width and spacing effect a 50 to 60 percent reduction in flow width and the minimum tailwater depth is not less than $2.15d_c$.

8. Design Discharge. Design discharge for the drop structure should be computed using the equation

$$Q = CLH^{3/2} \tag{8}$$

where

Q = design discharge, cfs
C = discharge coefficient = 3.0*
L = length of the drop structure crest, ft
H = energy head on the crest, ft

The length L of the weir should effect optimum use of channel cross section upstream. A trial-and-error procedure should be used to balance the crest height and width with the channel cross section.

9. Riprap Protection. Riprap protection should be provided immediately upstream and downstream of each structure. It is recommended that design criteria given in HDC 712-1 be used to meet stilling requirements and that given in EM 1110-2-1601 (reference 7) for upstream protection.

10. References.

(1) Etcheverry, B. A., Irrigation Practice and Engineering. 1st ed., Chapter VII, McGraw-Hill Book Company, New York, N. Y., 1916.

(2) Morris, B. T. and Johnson, D. C., "Hydraulic design of drop structures for gully control." Transactions, American Society of Civil Engineers, vol 108 (1943), pp 887-940.

(3) Vanoni, V. A. and Pollak, R. E., Experimental Design of Low Rectangular Drops for Alluvial Flood Channels. Report No. E-82, California Institute of Technology, Pasadena, Calif., September 1959.

(4) Donnelly, C. A. and Blaisdell, F. W., Straight Drop Spillway Stilling Basin. Technical Paper No. 15, Series B, St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, Minn., November 1954.

(5) U. S. Army Engineer Waterways Experiment Station, CE, Drop Structure for Gering Valley Project, Scottsbluff County, Nebraska, Hydraulic Model Investigation, by T. E. Murphy. Technical Report No. 2-760, Vicksburg, Miss., February 1967.

(6) U. S. Department of Agriculture, Soil Conservation Service, Engineer-Handbook, Drop Spillways. Section 11, Type C, Washington, D. C., p 5-11.

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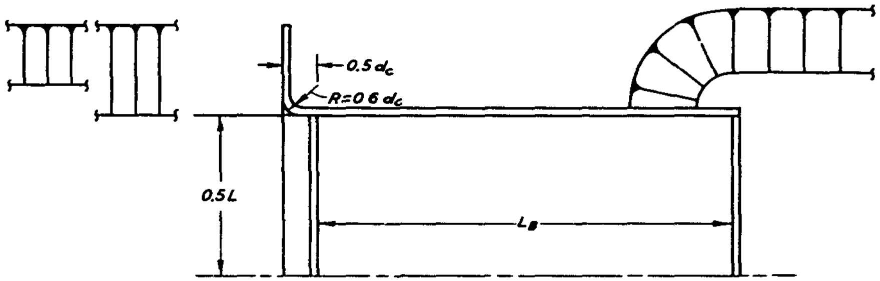
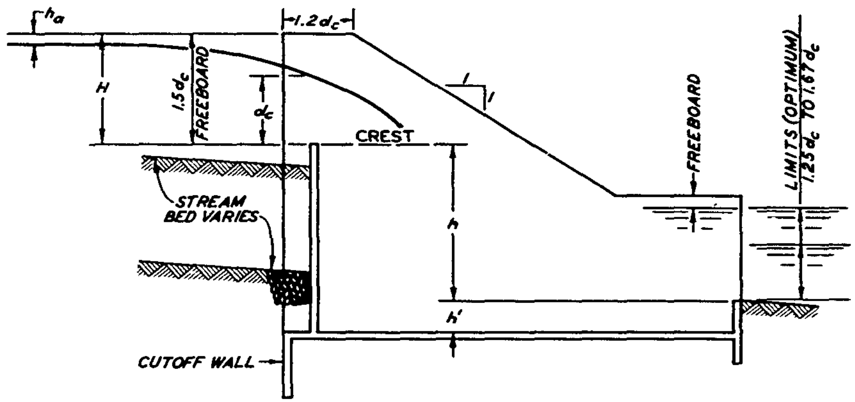
* Reduced for submergence effects when applicable.

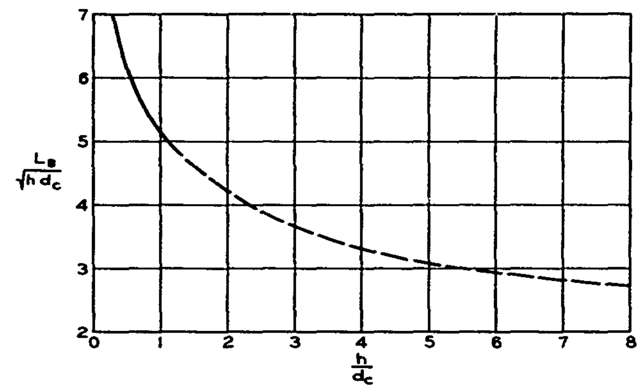
(7) U. S. Army, Office, Chief of Engineers, Engineering and Design; Hydraulic Design of Flood Control Channels. Engineer Manual EM 1110-2-1601, Washington, D. C., 1 July 1970.



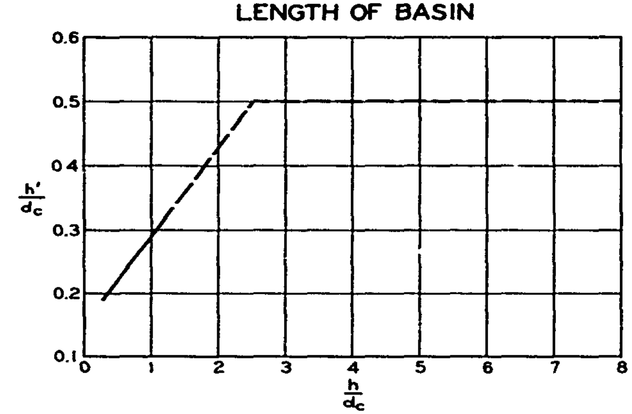
HALF PLAN



CENTER-LINE SECTION



LENGTH OF BASIN



END SILL HEIGHT

NOTE:

d_c=CRITICAL DEPTH OVER CREST
h =HEIGHT OF DROP
h' =HEIGHT OF END SILL
H =HEAD ON WEIR= $\frac{3}{2}$ (d_c)
h_a=VELOCITY HEAD
L_B=LENGTH OF BASIN
L =LENGTH OF WEIR CREST

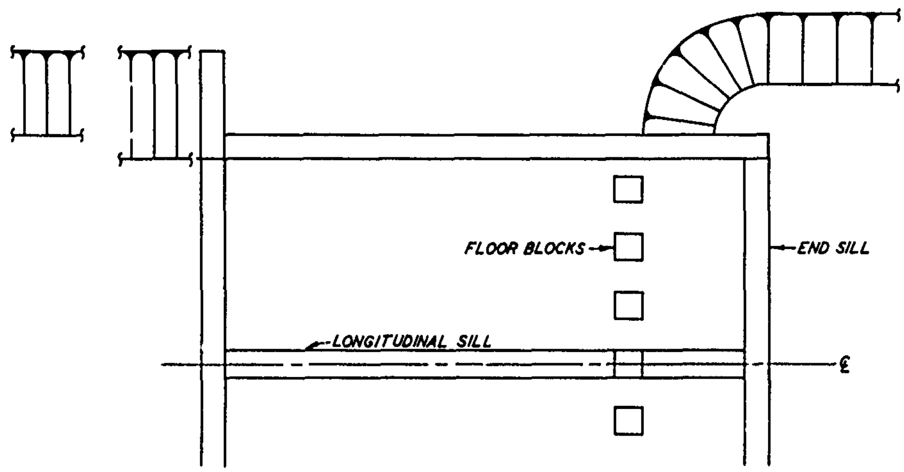
EXTRAPOLATED PORTIONS OF THE CURVES ARE NOT RECOMMENDED FOR LARGE STRUCTURES

SUBCRITICAL OPEN CHANNEL FLOW

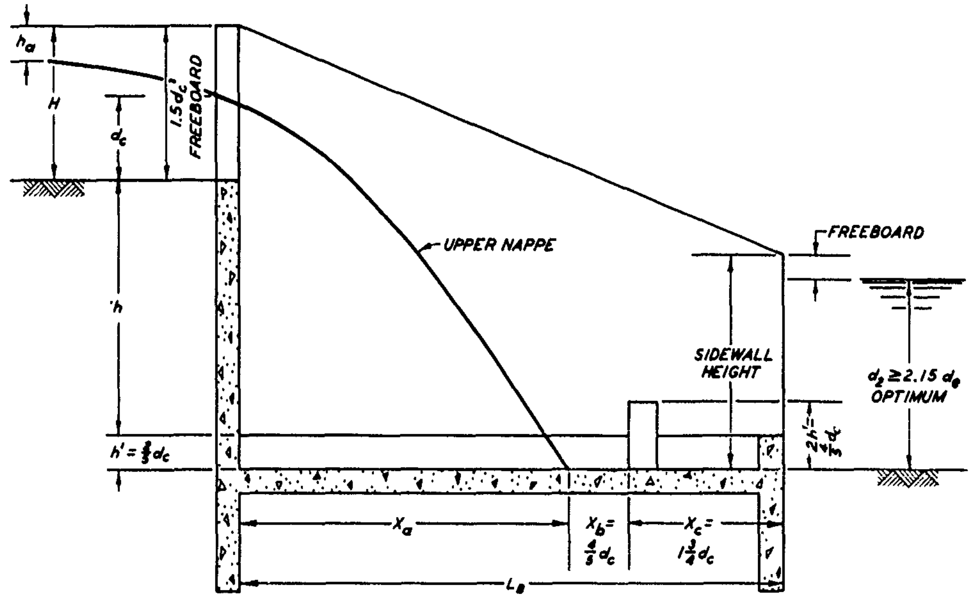
CIT-TYPE DROP STRUCTURE

HYDRAULIC DESIGN CHART 623

WES 7-73



HALF PLAN



CENTER-LINE SECTION

NOTE:

H = HEAD ON WEIR = $\frac{3}{2}(d_c)$

h_a = VELOCITY HEAD

d_2 = TAILWATER DEPTH

d_c = CRITICAL DEPTH OVER CREST

h = HEIGHT OF DROP

h' = HEIGHT OF END SILL

L_B = LENGTH OF STILLING BASIN = $X_a + X_b + X_c$

X_a = HORIZONTAL DISTANCE FROM CREST TO INTERSECTION OF UPPER NAPPE AND STILLING BASIN FLOOR

X_b = HORIZONTAL DISTANCE FROM INTERSECTION OF UPPER NAPPE AND STILLING BASIN FLOOR TO UPSTREAM FACE OF FLOOR BLOCKS

X_c = HORIZONTAL DISTANCE FROM UPSTREAM FACE OF FLOOR BLOCKS TO END OF STILLING BASIN

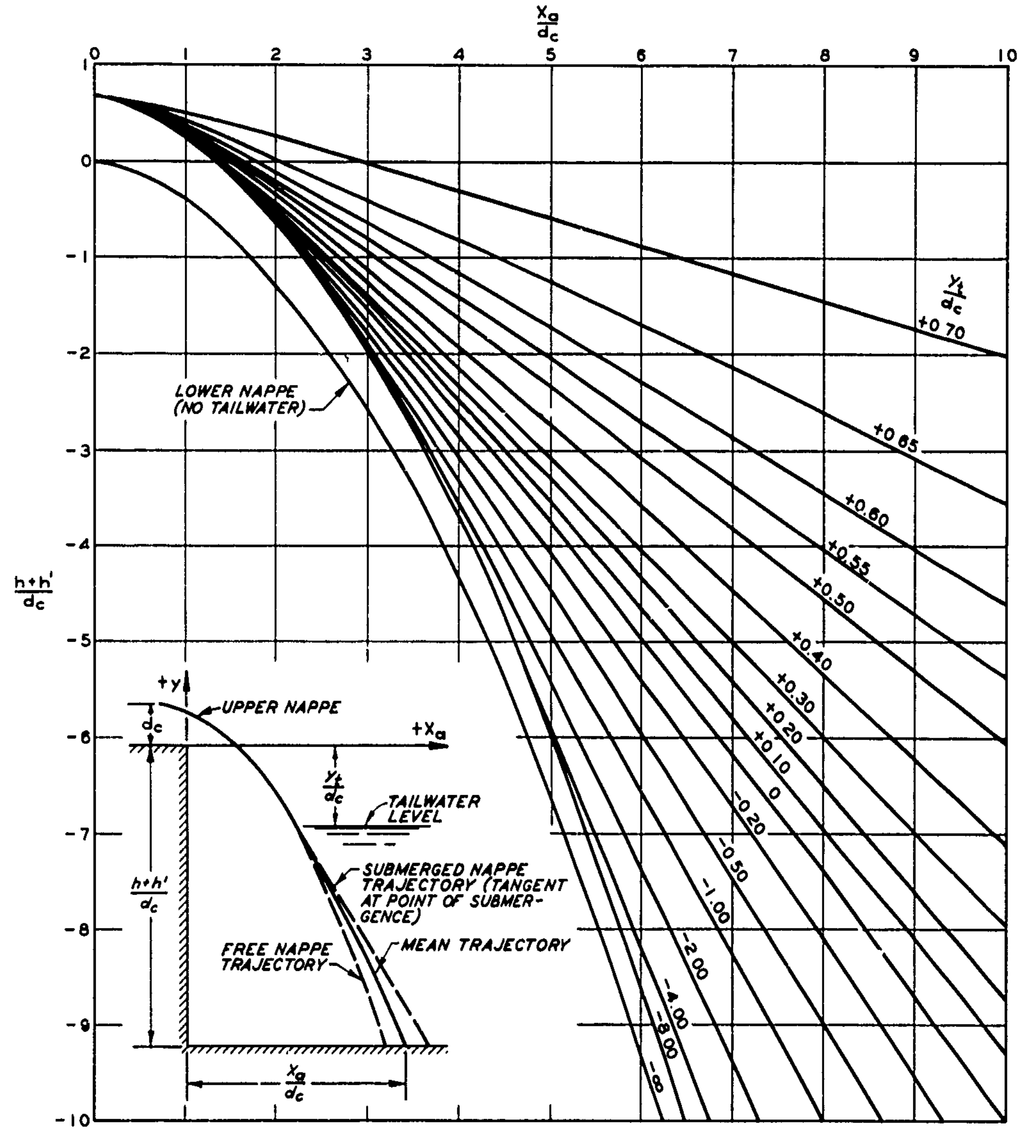
REDRAWN FROM FIG 10, REFERENCE 4.

SUBCRITICAL OPEN CHANNEL FLOW

SAF-TYPE DROP STRUCTURE BASIC GEOMETRY

HYDRAULIC DESIGN CHART 624

WES 7-73



NOTE:

d_c = CRITICAL DEPTH OVER CREST

h = HEIGHT OF DROP

h' = HEIGHT OF END SILL

X_a = HORIZONTAL DISTANCE FROM CREST TO INTERSECTION OF UPPER NAPPE AND STILLING BASIN FLOOR

Y_t = VERTICAL DISTANCE FROM CREST TO TAILWATER SURFACE (Y_t IS POSITIVE WHEN TAILWATER SURFACE IS ABOVE THE CREST, NEGATIVE WHEN TAILWATER SURFACE IS BELOW CREST)

REDRAWN FROM FIG. 2, REFERENCE 4

SUBCRITICAL OPEN CHANNEL FLOW

SAF-TYPE DROP STRUCTURE JET IMPACT LOCATION

HYDRAULIC DESIGN CHART 624-1

WES 7-73

HYDRAULIC DESIGN CRITERIA

SHEETS 631 TO 631-2

OPEN CHANNEL FLOW

RESISTANCE COEFFICIENTS

1. General. Because of its simplicity, the Manning equation has been used extensively in the United States in the evaluation of resistance losses in open channel flow. A comprehensive summary of the use of this equation in channel design is given in reference 1. Flow data and Manning's n's for 50 natural streams, together with color photographs of the channels, have also been published.[2] The Chezy equation[1] includes a resistance coefficient term that is applicable to all flow conditions. Hydraulic Design Chart 631 presents a general resistance diagram relating Chezy's C, Reynolds number, and relative roughness. The chart is useful in open channel flow problems.

2. Laboratory and field investigations have shown that the resistance coefficient varies with Reynolds numbers as well as with boundary surface roughness. Keulegan[3] has demonstrated that the Von Karman-Prandtl smooth and rough pipe resistance equations based on the Nikuradse test data can be applied to open channel flow with only minor adjustments in the equation constants. A recent ASCE progress report[4] recommends a Moody-type diagram for use in open channel flow, especially for flows in which the viscous effects are important.

3. Chezy Equation. The Chezy equation is

$$V = C\sqrt{RS}$$

where

V = mean channel velocity, ft per sec

C = Chezy resistance coefficient which is a function of Reynolds number and relative roughness of channel

R = hydraulic radius of channel, ft

S = slope of energy gradient

4. Resistance Coefficient Relations. The Darcy resistance coefficient f (see Hydraulic Design Chart 224-1) is defined as

$$f = \frac{8RSg}{V^2}$$

where g = acceleration of gravity.

The relation between C and f is

$$C = \sqrt{\frac{8g}{f}}$$

Similarly, the relation of C and n can be shown to be

$$C = \frac{1.486R^{1/6}}{n}$$

5. Effects of Reynolds Number. The Chezy resistance coefficient C is plotted as a function of Reynolds number in Chart 631. An auxiliary scale of Darcy resistance coefficient f is also shown for alternative use by the designer. The method of plotting is a form of the Moody diagram (Sheet 224-1). The resistance equations for smooth and rough flow based on Keulegan's results and recommended by Chow[1] are given and plotted in Chart 631. The rough flow limit based on Rouse's pipe flow criterion[5] is also shown. The Keulegan constants were used in the Colebrook-White equation (Chart 224-1) for the transition flow zone. The Reynolds number used for plotting is

$$R_e = \frac{4VR}{\nu}$$

where ν = the kinematic viscosity.

The use of this form of the Reynolds number is recommended in the ASCE task force report.[4]

6. Basic Data. The plotted data in Chart 631 are for concrete-lined channels. Both tranquil- and rapid-flow data are presented. The tranquil-flow data were computed from U. S. Army Engineer Waterways Experiment Station (WES) laboratory tests in brushed-concrete flumes[6,7] and from field tests results compiled by Scobey.[8,9] More recently obtained U. S. Bureau of Reclamation (USBR)[10] and Italian[11] field data have also been included. These data were selected on the basis of accuracy of flow measurements and conditions of concrete channel lining. Tests at the University of Iowa[12] indicate that the energy loss in flows having Froude numbers greater than 1.6 becomes a function of the Froude number and density and size of roughness elements. Additional energy loss is caused by instability of the flow. The plotted data points based on prototype tests at the Fort Randall[13] and Fort Peck[14] spillway chutes are for rapid flow with Froude numbers exceeding the stability criterion. These data represent the only known available measurements at R_e numbers approaching 10^8.

7. Suggested Design Criteria.

a. Resistance coefficients. The data plotted in Chart 631 can be used for guidance in the design of concrete-lined channels with subcritical velocities. Resistance coefficients for these channels generally are in the transition zone shown in the chart. The flow regime is seldom hydraulically smooth or fully rough and the resistance coefficient is usually a function of both the Reynolds number and the relative roughness. Chart 631-1 is a plot relating Chezy C, Manning's n, the equivalent roughness k_s, and the hydraulic radius. Theoretically it is only applicable to rough flow conditions. This chart should be useful for relating C and n for the design of channels with riprapped banks (Charts 631-4 and 631-4/1). The equation for n on Chart 631-1 was developed by solving the rough flow equation given in Chart 631 in terms of Manning's n.

b. Equivalent roughness k_s. In the use of Chart 631, a value of k_s (equivalent sand grain diameter) has to be specified for the prediction of resistance. The hydraulic roughness k_s in pipe flow is dependent only on the type of construction or the surface finish specified. However, in open channel flow it includes the effects of secondary flow resulting from boundary geometry and to a lesser extent the free water surface. Experimental data for correlation of surface texture, channel geometry, and the resulting hydraulic equivalent roughness k_s are very limited. However, considerable variation in the selected k_s value results in only small changes in the flow energy loss.

(1) The following tabulation presents average k_s values resulting from different types of concrete forming and surface finishing. It is based on computations made from the open channel resistance data plotted in Chart 631.

Average k_s, ft	Concrete Surface Finish
0.0006	18-year-old, 10-ft-wide rectangular aqueduct. Troweled sides and float-finished bottom (ref 9)
0.002	Laboratory rectangular and trapezoidal channels, brushed concrete finish (refs 6 and 7). Field channels, smooth, troweled cement finish (refs 8, 9, and 11)

(Continued)

Average k_s, ft	Concrete Surface Finish
0.003	10- to 20-year-old, 8- to 50-ft-wide trapezoidal channels constructed with modern rail-mounted slip traveling forms (ref 10)
0.005	Screed-finished spillway chute blocks with transverse joints at 20- to 25-ft intervals (refs 13 and 14)

(2) The tabulation above can be used for selecting design k_s values if the concrete forming and surface finishing can be obtained with good assurance. For general design computations the following k_s values for concrete are suggested:

Design Problem	Suggested k_s Value, ft
Discharge capacity	0.007
Maximum velocity	0.002
Proximity to critical depth*	
Subcritical flow	0.002
Supercritical flow	0.007

* To prevent undesirable undulating waves, flow-depth-to-critical depth ratios between 0.9 and 1.1 should be avoided.

(3) The determination of the equivalent surface roughness for riprap channels, rubble masonry, or other large roughness protrusions should be based on some estimate of the mean protrusion, riprap, or rock size. Use of the D_{50} (mean) size as k_s, based on equivalent sphere weight, is a good approximation for stone riprap.

8. Application. Chart 631-2 is a sample computation sheet illustrating the use of Charts 631 and 631-1.

9. References.

(1) Chow, V. T., Open-Channel Hydraulics. McGraw-Hill Book Co., Inc., New York, N. Y., 1959, pp 109-123.

(2) U. S. Geological Survey, Roughness Characteristics of Natural Channels, by H. H. Barnes, Jr. Water-Supply Paper 1849, Washington, D. C., 1967.

(3) Keulegan, G. H., "Laws of turbulent flow in open channels." Journal of Research, National Bureau of Standards, vol 21, Research Paper No. 1151 (December 1938), pp 707-741.

(4) Progress Report of the Task Force on Friction Factors in Open Channels, "Friction factors in open channels." ASCE, Hydraulics Division, Journal, vol 89, HY 2, paper 3464 (March 1963), pp 97-143.

(5) Rouse, H., Engineering Hydraulics; Proceedings of the Fourth Conference, Iowa Institute of Hydraulic Research, June 12-15, 1949. John Wiley & Sons, Inc., New York, N. Y., 1950, p 404.

(6) U. S. Army Engineer Waterways Experiment Station, CE, Roughness Standards for Hydraulic Models; Study of Finite Boundary Roughness in Rectangular Flumes, by Irene E. Miller and Margaret S. Peterson. Technical Memorandum No. 2-364, Report 1, Vicksburg, Miss., June 1953.

(7) __________, Hydraulic Capacity of Meandering Channels in Straight Floodways; Hydraulic Model Investigation, by E. B. Lipscomb. Technical Memorandum No. 2-429, Vicksburg, Miss., March 1956.

(8) U. S. Department of Agriculture, The Flow of Water in Flumes, by F. C. Scobey. Technical Bulletin No. 393, Washington, D. C., December 1933.

(9) __________, The Flow of Water in Irrigation and Similar Canals, by F. C. Scobey. Technical Bulletin No. 652, Washington, D. C., February 1939.

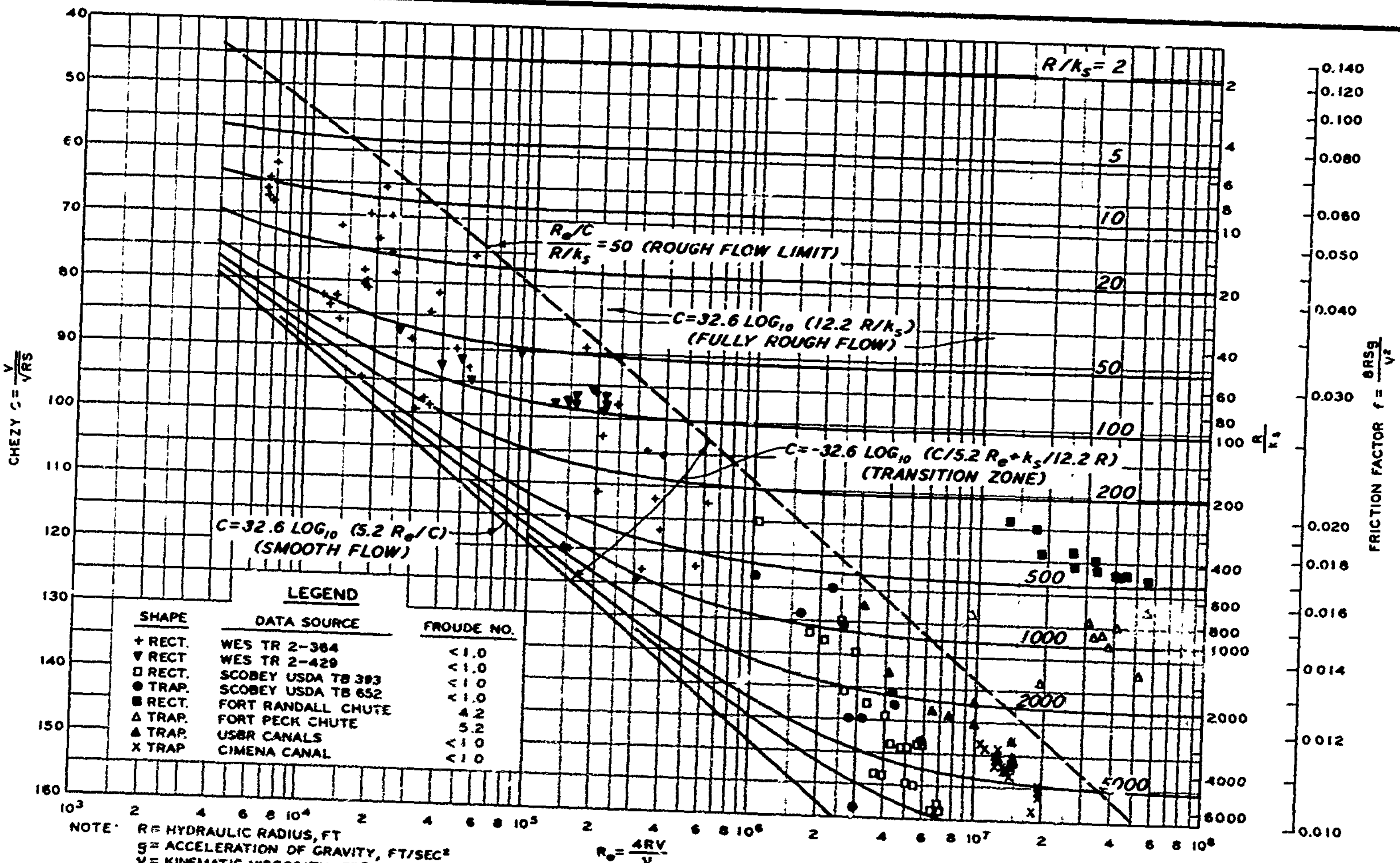
(10) U. S. Bureau of Reclamation, Analyses and Descriptions of Capacity Tests in Large Concrete-Lined Canals, by P. J. Tilp and M. W. Scrivner. Technical Memorandum 661, Denver, Colo., April 1964.

(11) Grassino, R., "Determination of roughness coefficients for Cimena Canal." L'Energia Elettrica, vol XL, No. 6 (June 1963), pp 429-436. Translation by Jan C. Van Tienhoven for U. S. Army Engineer Waterways Experiment Station, CE, Translation No. 65-3, Vicksburg, Miss., May 1965.

(12) Rouse, H., Koloseus, H. J., and Davidian, J., "The role of the Froude number in open-channel resistance." Hydraulic Research, Journal of the International Association for Hydraulic Research, vol 1, No. 1 (1963), pp 14-19.

(13) U. S. Army Engineer Waterways Experiment Station, CE, Flow in Chute Spillway at Fort Randall Dam; Hydraulic Prototype Tests, by C. J. Huval. Technical Report No. 2-716, Vicksburg, Miss., April 1966.

(14) U. S. Army Engineer District, Omaha, Nebraska. (Unpublished memorandum on Fort Peck Spillway tests, 1951.)



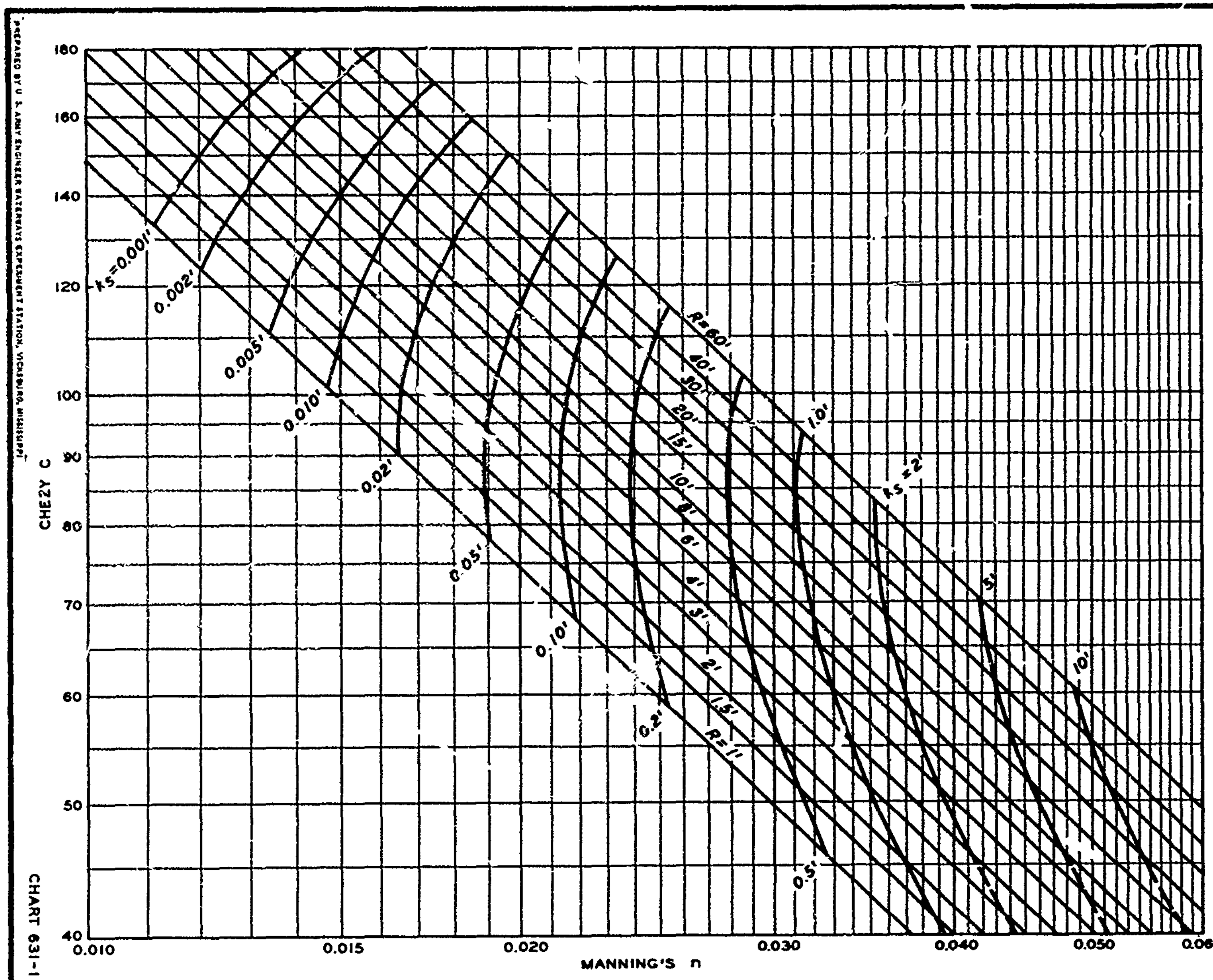
NOTE: R = HYDRAULIC RADIUS, FT
g = ACCELERATION OF GRAVITY, FT/SEC²
ν = KINEMATIC VISCOSITY, FT²/SEC
k_s = EFFECTIVE ROUGHNESS HEIGHT, FT
V = VELOCITY, FT/SEC
S = ENERGY GRADIENT SLOPE
R_e = REYNOLDS NUMBER

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

OPEN CHANNEL FLOW
RESISTANCE COEFFICIENTS

HYDRAULIC DESIGN CHART 631

REV 1-68 WES 8-58



BASIC EQUATIONS

$$C = 32.6 \log_{10} 12.2R/k_s$$

$$n = \frac{R^{1/6}}{23.85 + 21.95 \log_{10} R/k_s}$$

WHERE:

C = CHEZY COEFFICIENT
n = MANNING'S RESISTANCE COEFFICIENT
R = HYDRAULIC RADIUS, FT
k_s = EFFECTIVE ROUGHNESS HEIGHT, FT

OPEN CHANNELS

C-n-R-k_s RELATION

HYDRAULIC DESIGN CHART 631-1

REV 1-68 WES 9-58

GIVEN:

Concrete-lined channel
Shape, trapezoidal
Invert slope (S) = 0.0004
Flow depth (D) = 12 ft
Side slope = 1 on 2
Water temperature = 60 F
Discharge (Q) = 15,000 cfs
Construction, rail-mounted traveling forms

REQUIRED:

Equivalent roughness k_s
Chezy C
Base width B
Froude No. < 0.85
Check Manning's n

From tabulation of equivalent roughness (par. 7b(1), Sheets 631 to 631-2), $k_s = 0.003$ ft
From Chart 001-1, $\nu = 1.22 \times 10^{-5}$ ft^2/sec at 60 F

TRIAL COMPUTATIONS

1. Assume base width B = 50 ft

$$V = \frac{Q}{Area} = \frac{15{,}000}{74 \times 12} = 16.9 \text{ ft/sec}$$

Hydraulic radius $R = \frac{Area}{Wetted\ Perimeter} = \frac{74 \times 12}{103.6} = 8.57$ ft

$$R_e = \frac{4VR}{\nu} = \frac{4(16.9)(8.57)}{1.22 \times 10^{-5}} = 4.75 \times 10^7$$

$$\frac{R}{k_s} = \frac{8.57}{0.003} = 2860 \qquad C = 148 \text{ (Chart 631)}$$

$$V = C\sqrt{RS} = 148\sqrt{8.57 \times 0.0004} = 8.67 \text{ ft/sec} < 16.9 \text{ ft/sec}$$

2. Assume base width B = 110 ft

$$V = \frac{15{,}000}{134 \times 12} = 9.33 \text{ ft/sec} \qquad R = \frac{134 \times 12}{163.6} = 9.83 \text{ ft}$$

$$R_e = \frac{4(9.33)(9.83)}{1.22 \times 10^{-5}} = 3.0 \times 10^7$$

$$\frac{R}{k_s} = \frac{9.83}{0.003} = 3280 \qquad C = 149 \text{ (Chart 631)}$$

$$V = 149\sqrt{9.83 \times 0.0004} = 9.34 \approx 9.33 \text{ ft/sec}$$

3. Check Froude No. (F) and Manning's n

$$F = \frac{V}{\sqrt{gD}} \text{ (wide channel)} = \frac{9.33}{\sqrt{g(12)}} = 0.48 < 0.85$$

$$n = 0.0145 \text{ (Chart 631-1)}$$

OPEN CHANNEL FLOW
RESISTANCE COEFFICIENTS
SAMPLE COMPUTATION
HYDRAULIC DESIGN CHART 631-2

HYDRAULIC DESIGN CRITERIA

SHEETS 631-4 AND 631-4/1

OPEN CHANNEL FLOW

COMPOSITE ROUGHNESS

EFFECTIVE MANNING'S n

1. Tables of recommended roughness coefficients for use in the Manning formula for the solution of open channel flow problems have been published in references 1 and 2. Chow[2] includes recommended values for channels having different bed and bank materials. In wide, shallow channels the bed roughness effects predominate. Conversely, in narrow deep channels the bank roughness is the primary factor contributing to the flow energy losses.

2. Basic Data. Procedures for computing the effective roughness coefficient n to be used in the Manning formula for channels with different bed and bank roughnesses have been developed by Horton,[3] Colebatch,[4] Einstein,[5] and the U. S. Army Engineer District, Los Angeles, California.[6] In each case the effective n value is a function of the bed and bank roughnesses and their respective segments of the wetted perimeter or flow area. In their simplest form, the equations for effective n values can be written as

$$n_{eff} = \frac{\Sigma nA}{\Sigma A} \qquad \text{(Los Angeles District)} \quad (1)$$

$$n_{eff} = \left[\frac{\Sigma(n^{3/2} P)}{P}\right]^{2/3} \qquad \text{(Horton or Einstein)} \quad (2)$$

$$n_{eff} = \left[\frac{\Sigma(n^{3/2} A)}{\Sigma A}\right]^{2/3} \qquad \text{(Colebatch)} \quad (3)$$

A and P are the channel flow subareas and wetted perimeter segments, respectively; n is the respective Manning roughness coefficient for each segment considered.

3. Study of the equations given in paragraph 2 indicates that for channels with smooth inverts and rough banks, use of the Horton-Einstein equation results in more conservative design than use of either the Colebatch or the Los Angeles District equation. Laboratory and field investigations are needed for complete evaluation of the equations. The use of the Horton-Einstein equation is suggested for design purposes pending availability of additional test data.

4. For rectangular or trapezoidal channels, equation 2 can be written in the form

$$n_{eff} = \left(\frac{n_1^{3/2} P_1 + 2n_2^{3/2} P_2}{P_1 + 2P_2} \right)^{2/3} \tag{4}$$

where the subscripts 1 and 2 refer to the bed and bank wetted perimeters, respectively. The terms are further defined in the sketch in Hydraulic Design Chart 631-4/1.

5. Application. Chart 631-4 provides a rapid graphical method for determining the solution of equation 2 to obtain an effective n value for use in the design of uniform channel sections with different bed and bank roughnesses. The ordinates of the chart indicate the bed, bank, and combined effective roughness coefficients. The abscissas are values of the ratio of the bed and bank wetted perimeters. The effective n value is determined in the following manner. The chart is entered vertically from the bottom with the given value of $2P_2/P_1$ to its intersection with an imaginary line connecting n_1 and n_2. The value of n_{eff} at this point is read on the right side of the chart.

6. Chart 631-4/1 can be used to obtain the required wetted perimeter ratio for use with Chart 631-4. Chart 631-4/1 presents bank-bed wetted perimeter relations for trapezoidal and rectangular channel sections as functions of the bed width, flow depth, and bank slope. These charts can be used with Charts 631 and 631-1 for the design of channels with riprapped banks.

7. References.

(1) King, H. W., Handbook of Hydraulics for the Solution of Hydraulic Problems, revised by E. F. Brater, 4th ed. McGraw-Hill Book Co., Inc., New York, N. Y., 1954, Table 76, p 20.

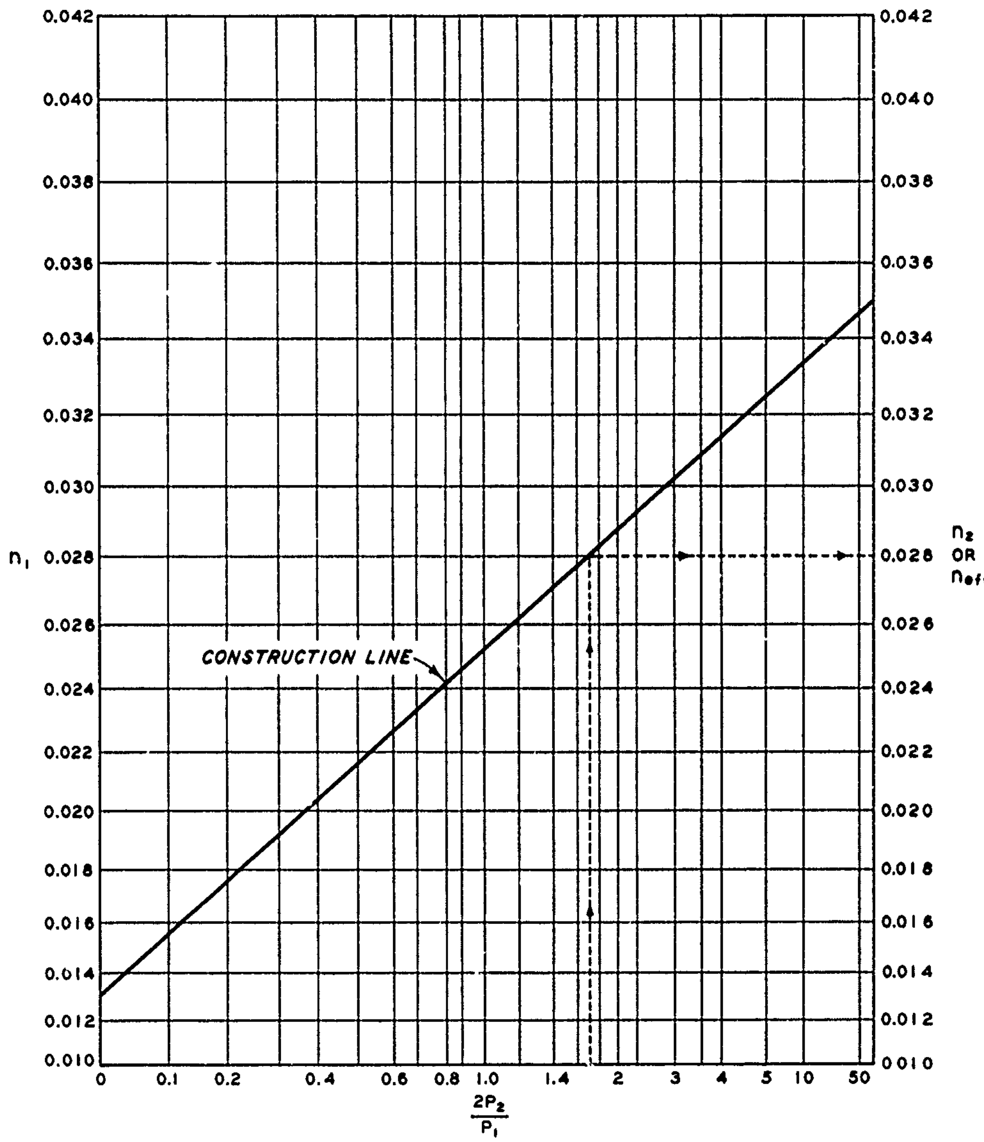
(2) Chow, V. T., Open-Channel Hydraulics. McGraw-Hill Book Co., Inc., New York, N. Y., 1959, Tables 5 and 6, p 111.

(3) Horton R. E., "Separate roughness coefficients for channel bottom and sides." Engineering News-Record, vol iii, No. 22 (30 November 1933), pp 652-653.

(4) Colebatch, G. T., "Model tests on Liawenee Canal roughness coefficients." Transactions of the Institution, Journal of the Institution of Engineers, vol 13, No. 2, Australia (February 1941), pp 27-32.

(5) Einstein, H. A., "Der hydraulische oder Profil-Radius." Schweizerische Bauzeitung, vol 103, No. 8 (24 February 1934), pp 89-91.

(6) U. S. Army, Office, Chief of Engineers, Hydraulic Design of Flood Control Channels. EM 1110-2-1601 (unpublished Engineer Manual draft).



EFFECTIVE MANNING'S n (EQUATION 4)

$$n_{eff} = \left(\frac{n_1^{3/2} P_1 + 2 n_2^{3/2} P_2}{P_1 + 2P_2} \right)^{2/3}$$

WHERE.

n_1 = BED ROUGHNESS

n_2 = SIDE SLOPE ROUGHNESS

n_{eff} = EFFECTIVE ROUGHNESS

P_2 = SIDE SLOPE WALL LENGTH

P_1 = BOTTOM WIDTH

NOTE: GRAPH BASED ON FIGURE 8, REF 4

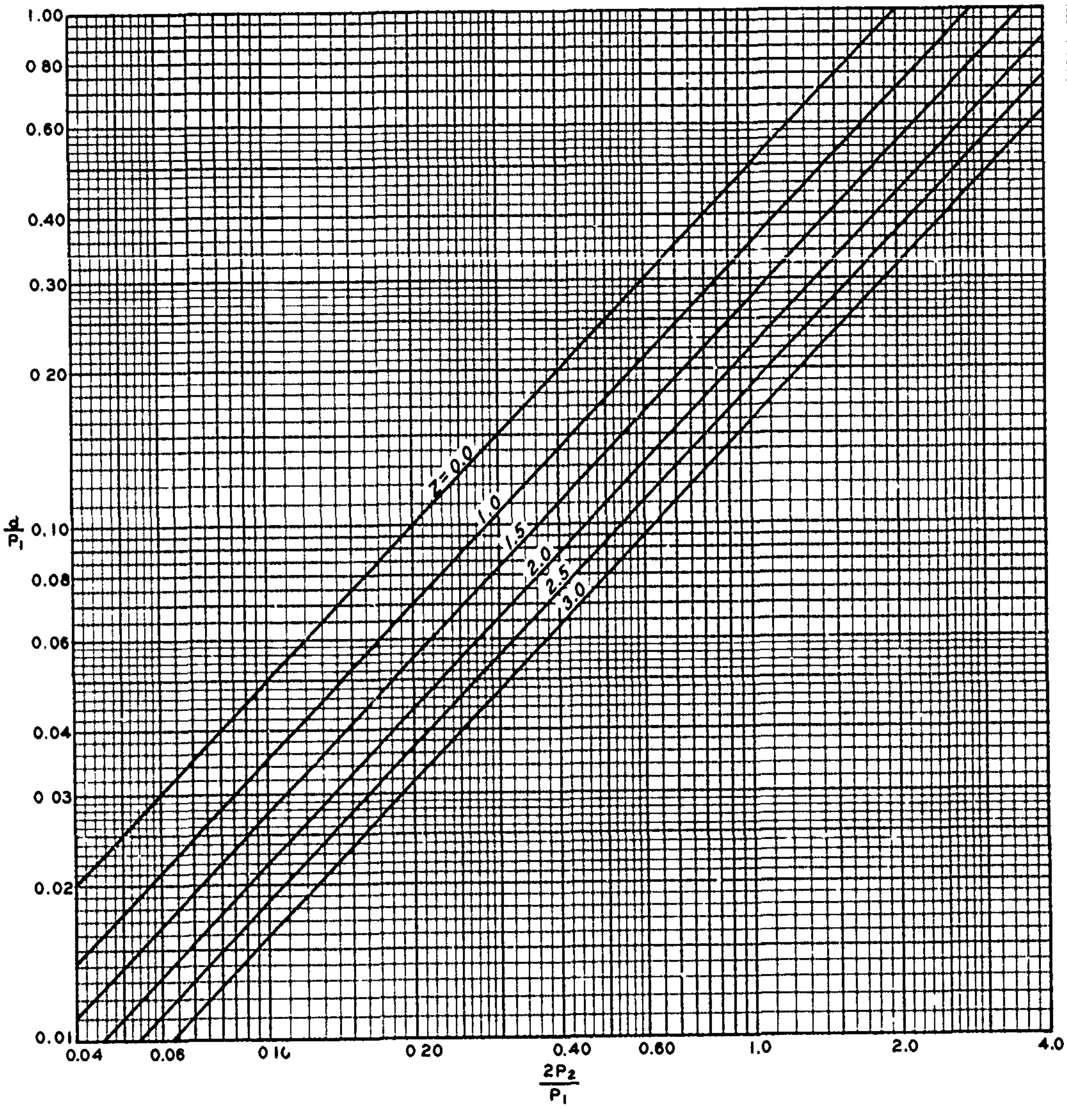
OPEN CHANNEL FLOW

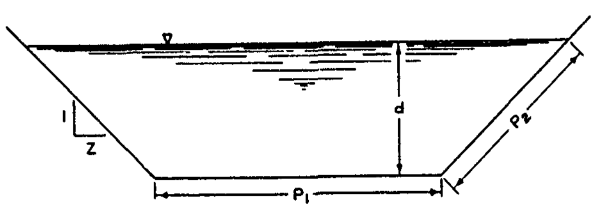
COMPOSITE ROUGHNESS EFFECTIVE MANNING'S n

HYDRAULIC DESIGN CHART 631-4

WES 1-68

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI





$P_2 = d\sqrt{1+Z^2}$

OPEN CHANNEL FLOW
COMPOSITE ROUGHNESS
WETTED PERIMETER RELATION

HYDRAULIC DESIGN CHART 631-4/1

WES 1-68

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

HYDRAULIC DESIGN CRITERIA

SHEET 660-1

CHANNEL CURVES

SUPERELEVATION

1. Purpose. Flows in curved channels result in increases in depth along the outside channel walls with corresponding decreases along the inside walls. The difference in the water-surface elevations between the channel center line and the outside wall is called the flow superelevation. This rise in water surface is a function of the channel shape, velocity, width, and radius of curvature. Chart 660-1 presents a graphical means of estimating superelevation for various combinations of channel velocities, widths, and radii of curvature.

2. Design Controls. Channel capacity (wall heights) should be based on the maximum expected resistance (friction) factor. The curve geometry and flow superelevation should be based on the minimum expected resistance factor. This design combination should result in economically conservative design for all flows.

3. Design Equations. The transverse rise in water surface of flow in a channel bend can be adequately described for both tranquil and rapid flow using an equation adapted from the centrifugal force equations.

$$\Delta y = C \frac{V^2 W}{gr} \tag{1}$$

where

Δy = the rise (superelevation plus surface disturbances) in water surface between the channel center line and the outside wall, ft

C = a coefficient depending upon flow Froude number, channel shape, and curve geometry

V = average channel velocity, fps

W = straight channel water-surface width, ft

g = acceleration of gravity, ft/sec^2

r = radius of curvature at center line, ft

The following tabulation relates the coefficient C with flow conditions, channel shape, and curve geometry. These relations are also shown by the sketches in Chart 660-1.

Type of Flow	Channel Shape	Curve Geometry	Coefficient C Value
Tranquil	Rect	Simple	0.5
Tranquil	Trap.	Simple	0.5
Rapid	Rect	Simple	1.0
Rapid	Trap.	Simple	1.0
Rapid	Rect	Spiral transition	0.5
Rapid	Trap.	Spiral transition	1.0
Rapid	Rect	Spiral-banked	0.5

4. Curve Design.

a. Tranquil flow. The required increase in the outer wall height in a channel curve over that of the straight channel for both rectangular and trapezoidal channels is obtained from Chart 660-1 using a C value of 0.5. The inner wall height should remain that of the straight channel. The unbalanced flow condition in the curve causes helicoidal flow that can result in undesirable scour and deposition in and downstream from the curve. Tests by Shukry[1] indicate that helicoidal flow can be minimized if the curve radius is greater than three times the channel width.

b. Rapid flow. Rapid flow in a simple circular curve results in a transverse rise in the water surface approximately twice that occurring with tranquil flow. This increase results from surface disturbances generated by changes in direction. These disturbances persist for many channel widths downstream of the curve. Superelevation for rapid flow can be estimated from Chart 660-1 using the appropriate C values given in the tabulation above or in the chart. A detailed analysis of the cross waves generated in simple curves is given by Ippen.[2]

The criterion for minimum radius of a simple curve, based on structures built by the Los Angeles District, is:

$$r_{min} = \frac{4V^2W}{gy} \quad (2)$$

with y equal to the flow depth for the minimum expected friction factor (Chart 631). This criterion is recommended for rapid flow curves with or without invert banking. A similar criterion for maximum allowable superelevation for acceptable flow conditions in rectangular channels is

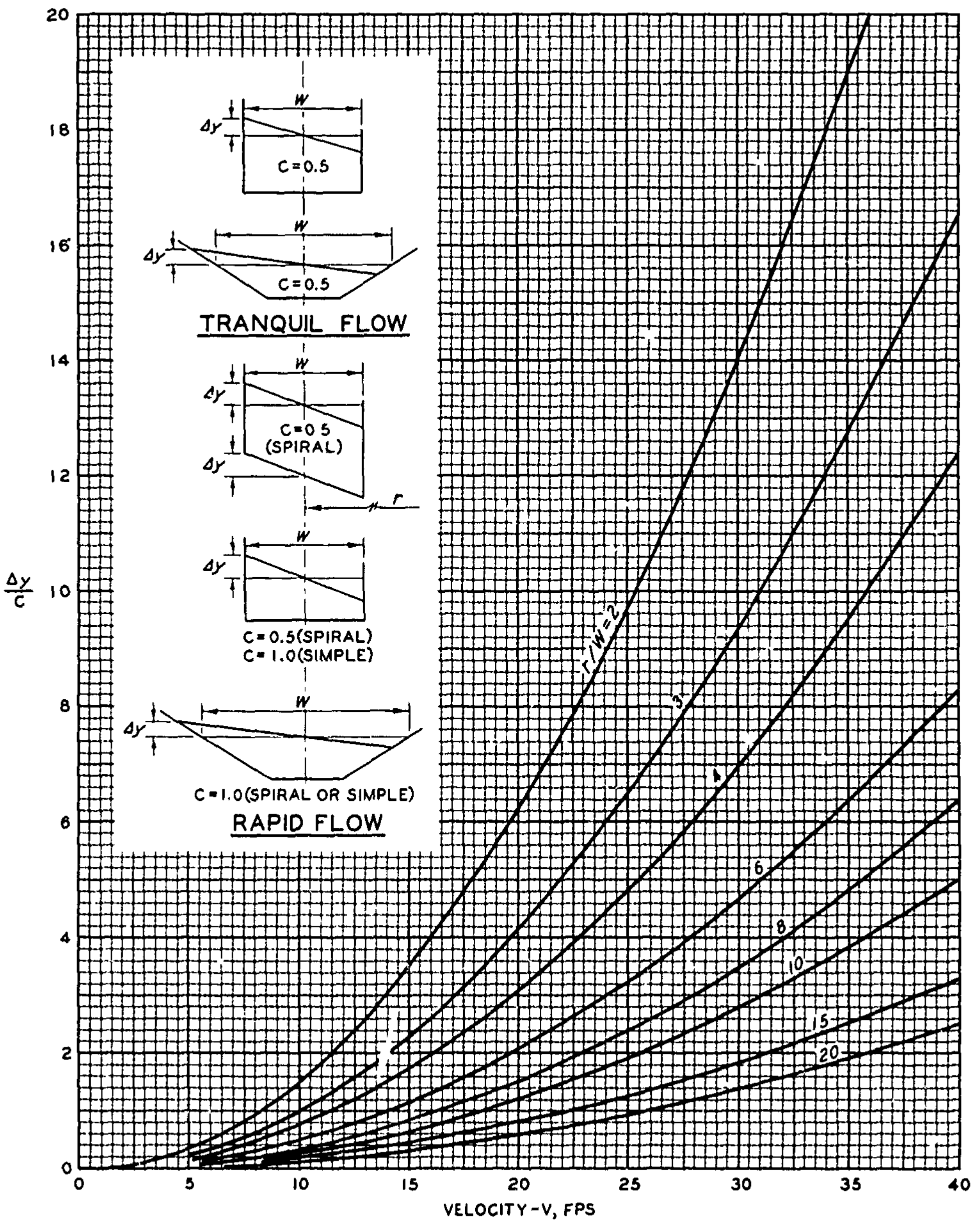
$$\Delta y_{max} = 0.09W \quad (3)$$

c. Invert banking. Invert banking maintains flow stability in curved channels and when used with spiral transitions results in minimum total rise in water surface between the channel center line and outside wall. It is limited to channels of rectangular cross sections. The invert is usually banked by rotating the bottom about the channel center line. The invert along the inside wall is depressed by Δy below the center-line elevation with a corresponding rise along the outside wall. The banking upstream and downstream from the curve should be accomplished linearly in accordance with the spiral transition lengths determined from equation 3 of Sheets 660-2 to 660-2/4. Wall heights on both sides of banked curves are usually designed to be the same as the wall height of the straight channel. Banking of trapezoidal channels is not practicable. Such channels should be designed wherever possible to have long radius curves resulting in minimum superelevation.

5. References.

(1) Shukry, A., "Flow around bends in an open flume." Transactions, American Society of Civil Engineers, vol 115, paper 2411 (1950), pp 751-779.

(2) Ippen, A. T., "Channel transitions and controls," Engineering Hydraulics, H. Rouse, ed. John Wiley & Sons, Inc., New York, N. Y., 1950, pp 496-588.



EQUATION

$$\Delta y = C \frac{V^2}{g} \frac{W}{r}$$

WHERE:

- V = AVERAGE VELOCITY
- Δy = SUPERELEVATION
- W = WATER SURFACE WIDTH (LEVEL)
- r = CURVE RADIUS
- C = CONSTANT
- g = GRAVITATIONAL ACCELERATION

CHANNEL CURVES
SUPERELEVATION

HYDRAULIC DESIGN CHART 660-1

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

WES 9-70

HYDRAULIC DESIGN CRITERIA

SHEETS 660-2 TO 660-2/4

CHANNEL CURVES WITH

SPIRAL TRANSITIONS

RAPID FLOW

1. Purpose. Spiral transitions are used to provide gradual change in channel curvature for rapid flow entering and leaving circular bends.[1] The compound circular curve has also been used for this purpose.[2] Use of spiral transitions eliminates the surface disturbances discussed in Sheet 660-1 and minimizes required wall height increases or channel banking.

2. Spiral Transitions. Spiral curves involve the solution of cubic equations by complex procedures, extensive successive approximation, or computers. The Los Angeles District (LAD) has prepared extensive spiral tables for easier manual design of rapid flow channels.[3] HDC 660-2 to 660-2/4 summarize these tables and illustrate their application to channel design.

3. The LAD spiral is a modification of Talbot's railroad spiral and consists of a series of compounded circular arcs of 12.5-ft lengths. The spiral has varying radii, decreasing in finite steps from the beginning of the spiral. The curve geometry, equations, and the definitions used to develop the LAD tables are given in Chart 660-2. Two equal spirals are shown, one upstream and one downstream of the circular curve. The central angle of the first arc (δ_1) establishes the shape of the spiral. The central angle subtended by a spiral of n number of arcs is given by:

$$\Delta s = n^2 \delta_1 \tag{1}$$

where

Δs = total central angle at the n^{th} arc of the spiral, sec

n = number of arc lengths of 12.5 ft each

δ_1 = central angle of the first arc, sec

4. Unbanked Curves. The minimum length of spiral recommended by Douma[4] for an unbanked curve is

$$L = 1.82 \frac{VW}{\sqrt{gy}} \tag{2}$$

where V and y are the velocity and flow depth, respectively, computed using a minimum resistance coefficient (Chart 631) and W is the water-surface width.

5. Banked Curves. The minimum spiral length recommended by Gildea and Wong[5] for banked curves is:

$$L = 30\Delta y \tag{3}$$

where Δy is the rise in water surface between the channel center line and the outside wall. Use of this criterion will not usually result in free drainage of a channel banked by rotating the invert about the center-line elevation.

6. Unequal Spirals. Unequal spiral lengths at the beginning and end of the circular curve may be required to meet special field conditions. The geometric relations between the spirals and the circular curve are given in Chart 660-2/1. With these relations determined, the design for each spiral proceeds as in the case of equal spirals.

7. Spiral Design Tables. The original LAD tables have been abridged and are presented in Chart 660-2/2. The chart should be adequate for design purposes and for preparation of contract drawings. Values of spiral lengths L, tangent distances X, and offsets Y are tabulated for n number of stations for 22 spirals. The method of computing values of X and Y, and the radius r of the central simple curve is given in reference 3. The curve number corresponds to the value of the first spiral arc angle δ_1, in sec, and indicates the rate of change in curvature. The minimum spiral length should be that which satisfies equation 2 (unbanked) or 3 (banked), provides optimum fit to local physical conditions, and is commensurate with economy of construction.

8. Application. The computation procedure for a banked invert curve with spiral transitions at each end is given in Chart 660-2/3. The final curve layout for the example is given in Chart 660-2/4. In cases of intermittent flow the banking may result in an undesirable pool of stagnant water along the inside wall. This can be avoided by selecting a longer downstream spiral. The length of this spiral is dependent upon the curve number selected and the number of spiral arc lengths required to attain a radius approximating that computed for the central curve. Twice the spiral length multiplied by the channel slope must equal or exceed the invert banking for free drainage.

9. Computer Program. A computer program for the design and field layout of the channel curve geometry is given in Appendix V of EM 1110-2-1601.[6]

10. References.

(1) U. S. Army Engineer District, Los Angeles, CE, Hydraulic Model Study, Los Angeles River Improvements, Whitsett Avenue to Tujunga Wash, July 1949.

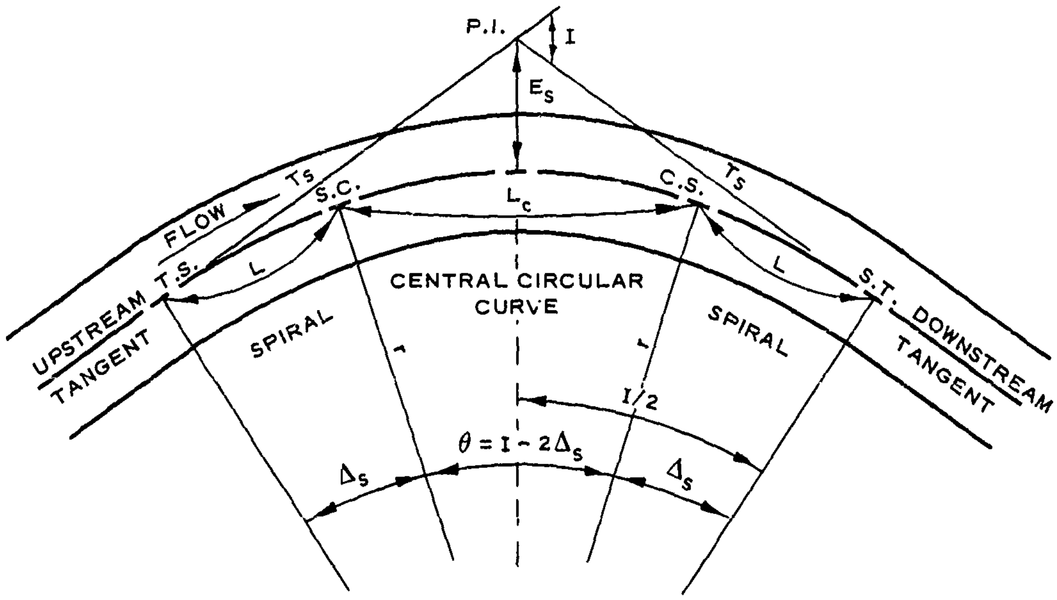
(2) Ippen, A. T., and Knapp, R. T., Experimental Investigations of Flow in Curved Channels. Reproduced by U. S. Army Engineer Office, Los Angeles, Calif. (2 volumes), 1958 (abstract of Results and Recommendations).

(3) U. S. Army Engineer District, Los Angeles, CE, Modified Spiral Curve Tables, June 1948.

(4) Douma, J. H., Discussion of "High-velocity flow in open channels; A symposium." Transactions, American Society of Civil Engineers, vol 116, paper 2434 (1951), pp 388-393.

(5) Gildea, A. P., and Wong, R. F., "Flood control channel hydraulics." Proceedings, Twelfth Congress of the International Association for Hydraulic Research, 11-14 September 1967, vol 1 (1967), pp 330-337.

(6) U. S. Army, Office, Chief of Engineers, "Appendix V: Computer program for designing banked curves for supercritical flow in rectangular channels," Engineering and Design; Hydraulic Design of Flood Control Channels. EM 1110-2-1601, Washington, D. C., 1 July 1970.



a. CHANNEL WITH SPIRAL CURVES

$$T_s = X - r\sin\Delta_s + (Y + r\cos\Delta_s)\tan\frac{I}{2}$$

$$E_s = \left[Y + r\sin\Delta_s\tan\left(\frac{I}{2} - \Delta_s\right)\right]\sec\frac{I}{2} + r\left[\sec\left(\frac{I}{2} - \Delta_s\right) - 1\right]$$

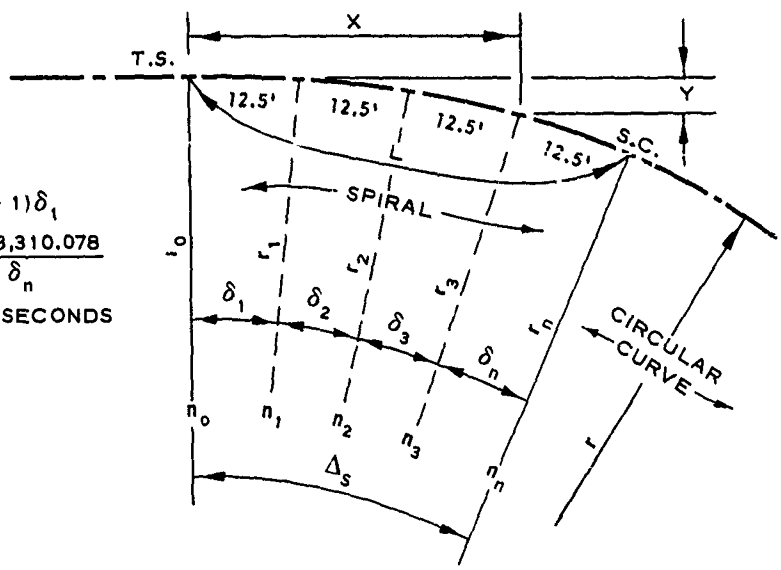
$$L_c = \frac{(I - 2\Delta_s)r}{57.2958}\ ;\ I,\ \Delta_s \text{ IN DEGREES}$$

$$\delta_n = (2n - 1)\delta_1$$

$$r_n = \frac{2{,}578{,}310.078}{\delta_n}$$

δ_1, δ_n IN SECONDS

$$\Delta_s = n^2\delta_1$$



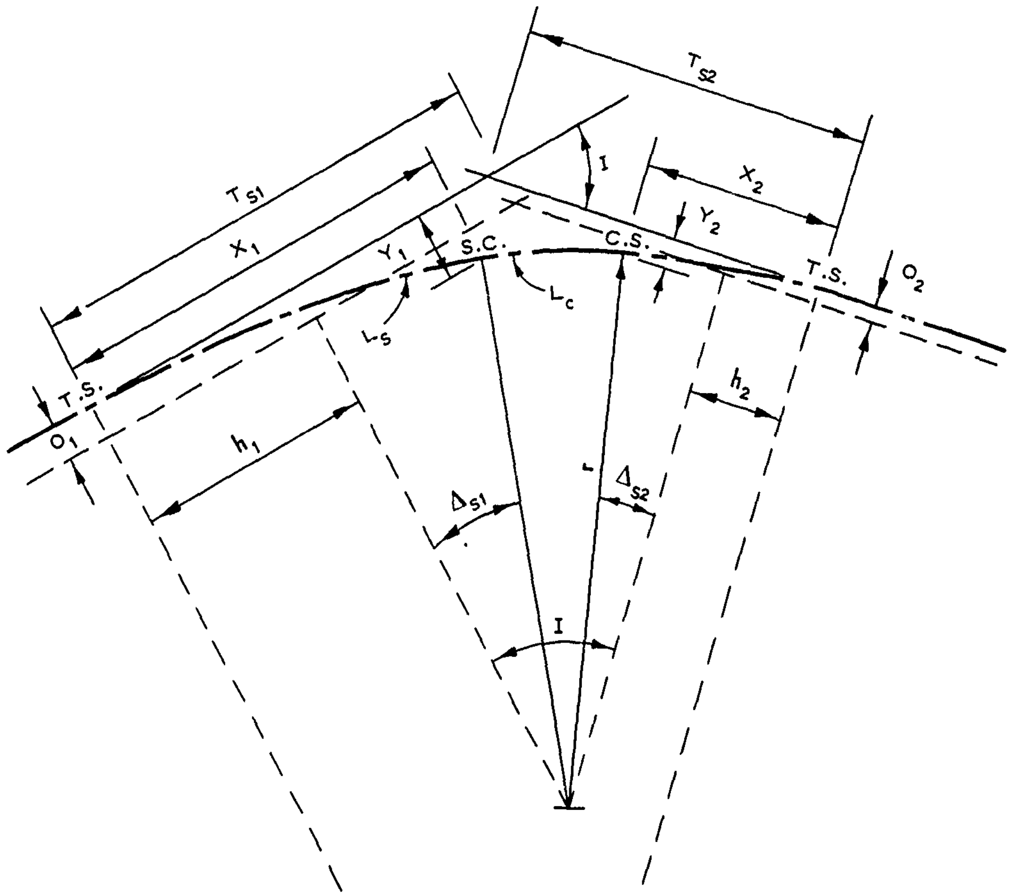
b. SPIRAL DETAILS

CHANNEL CURVE GEOMETRY
EQUAL SPIRALS

HYDRAULIC DESIGN CHART 660-2

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

WES 9-70



$$T_{S1} = \frac{(r + O_2) - (r + O_1)\cos I}{\sin I} + h_1$$

$$T_{S2} = \frac{(r + O_1) - (r + O_2)\cos I}{\sin I} + h_2$$

WHERE

$$h_1 = X_1 - r \sin \Delta_{S1}$$

$$h_2 = X_2 - r \sin \Delta_{S2}$$

$$O_1 = Y_1 - r(1 - \cos \Delta_{S1})$$

$$O_2 = Y_2 - r(1 - \cos \Delta_{S2})$$

NOTE: SEE CHART 660-2 FOR SPIRAL AND SIMPLE CURVE DETAILS

CHANNEL CURVE GEOMETRY

UNEQUAL SPIRALS

HYDRAULIC DESIGN CHART 660-2/1

No. 7 Curve

n	L, ft	r, ft	Δθ ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 00 07	12.500	0.0
2	25.0	92,078	00 00 28	25.000	0.001
3	37.5	61,386	00 01 03	37.500	0.004
4	50.0	46,039	00 01 52	50.000	0.009
5	62.5	36,831	00 02 55	62.500	0.018
6	75.0	30,693	00 04 12	75.000	0.031
7	87.5	26,300	00 05 43	87.500	0.049
8	100.0	23,020	00 07 28	100.000	0.073
9	112.5	20,462	00 09 27	112.500	0.104
10	125.0	18,416	00 11 40	125.000	0.142
11	137.5	16,742	00 14 07	137.500	0.189
12	150.0	15,346	00 16 48	150.000	0.245
13	162.5	14,166	00 19 43	162.499	0.312
14	175.0	13,154	00 22 52	174.999	0.389
15	187.5	12,277	00 26 15	187.499	0.478
16	200.0	11,510	00 29 52	199.998	0.580
17	212.5	10,833	00 33 43	212.498	0.696
18	225.0	10,231	00 37 48	224.997	0.826
19	237.5	9,692	00 42 07	237.496	0.971
20	250.0	9,208	00 46 40	249.995	1.133
21	262.5	8,769	00 51 27	262.494	1.311
22	275.0	8,371	00 56 28	274.993	1.507
23	287.5	8,007	01 01 43	287.491	1.722
24	300.0	7,673	01 07 12	299.989	1.956
25	312.5	7,366	01 12 55	312.486	2.211
26	325.0	7,083	01 18 52	324.983	2.487
27	337.5	6,821	01 25 03	337.479	2.785
28	350.0	6,577	01 31 28	349.975	3.106
29	362.5	6,350	01 38 07	362.470	3.451
30	375.0	6,139	01 45 00	374.965	3.820
31	387.5	5,941	01 52 07	387.459	4.214
32	400.0	5,755	01 59 28	399.952	4.635

No. 10 Curve

n	L, ft	r, ft	Δθ ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 00 10	12.500	0.0
2	25.0	64,450	00 00 40	25.000	0.002
3	37.5	42,966	00 01 30	37.500	0.006
4	50.0	32,225	00 02 40	50.000	0.013
5	62.5	25,780	00 04 10	62.500	0.026
6	75.0	21,483	00 06 00	75.000	0.044
7	87.5	18,414	00 08 10	87.500	0.070
8	100.0	16,112	00 10 40	100.000	0.104
9	112.5	14,322	00 13 30	112.500	0.148
10	125.0	12,890	00 16 40	125.000	0.203
11	137.5	11,718	00 20 10	137.500	0.270
12	150.0	10,742	00 24 00	149.999	0.350
13	162.5	9,915	00 28 10	162.499	0.445
14	175.0	9,207	00 32 40	174.998	0.556
15	187.5	8,593	00 37 30	187.498	0.683
16	200.0	8,056	00 42 40	199.997	0.829
17	212.5	7,582	00 48 10	212.496	0.994
18	225.0	7,161	00 54 00	224.994	1.180
19	237.5	6,784	01 00 10	237.493	1.387
20	250.0	6,445	01 06 40	249.991	1.618
21	262.5	6,138	01 13 30	262.488	1.873
22	275.0	5,859	01 20 40	274.985	2.153
23	287.5	5,604	01 28 10	287.481	2.460
24	300.0	5,371	01 36 00	299.977	2.795
25	312.5	5,156	01 44 10	312.471	3.159
26	325.0	4,958	01 52 40	324.965	3.553
27	337.5	4,774	02 01 30	337.458	3.979
28	350.0	4,604	02 10 40	349.949	4.437
29	362.5	4,445	02 20 10	362.440	4.929
30	375.0	4,297	02 30 00	374.929	5.456
31	387.5	4,158	02 40 10	387.416	6.020
32	400.0	4,028	02 50 40	399.901	6.621

No. 14 Curve

n	L, ft	r, ft	Δθ ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 00 14	12.500	0.0
2	25.0	46,039	00 00 56	25.000	0.003
3	37.5	30,693	00 02 06	37.500	0.008
4	50.0	23,020	00 03 44	50.000	0.019
5	62.5	18,416	00 05 50	62.500	0.036
6	75.0	15,346	00 08 24	75.000	0.062
7	87.5	13,154	00 11 26	87.500	0.098
8	100.0	11,510	00 14 56	100.000	0.146
9	112.5	10,231	00 18 54	112.500	0.207
10	125.0	9,208	00 23 20	124.999	0.284
11	137.5	8,371	00 28 14	137.499	0.378
12	150.0	7,673	00 33 36	149.999	0.490
13	162.5	7,083	00 39 26	162.498	0.623
14	175.0	6,577	00 45 44	174.997	0.778
15	187.5	6,139	00 52 30	187.496	0.957
16	200.0	5,755	00 59 44	199.994	1.161
17	212.5	5,416	01 07 26	212.492	1.392
18	225.0	5,115	01 15 36	224.989	1.652
19	237.5	4,846	01 24 14	237.486	1.942
20	250.0	4,604	01 33 20	249.982	2.265
21	262.5	4,385	01 42 54	262.477	2.622
22	275.0	4,185	01 52 56	274.970	3.014
23	287.5	4,003	02 03 26	287.463	3.444
24	300.0	3,837	02 14 24	299.954	3.913
25	312.5	3,683	02 25 50	312.444	4.422
26	325.0	3,541	02 37 44	324.932	4.974
27	337.5	3,410	02 50 06	337.417	5.569
28	350.0	3,289	03 02 56	349.901	6.211
29	362.5	3,175	03 16 14	362.382	6.900
30	375.0	3,069	03 30 00	374.860	7.638
31	387.5	2,970	03 44 14	387.335	8.427
32	400.0	2,877	03 58 56	399.807	9.268

No. 18 Curve

n	L, ft	r, ft	Δθ ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 00 18	12.500	0.001
2	25.0	35,810	00 01 12	25.000	0.003
3	37.5	23,873	00 02 42	37.500	0.010
4	50.0	17,905	00 04 48	50.000	0.024
5	62.5	14,324	00 07 30	62.500	0.046
6	75.0	11,937	00 10 48	75.000	0.080
7	87.5	10,231	00 14 42	87.500	0.126
8	100.0	8,952	00 19 12	100.000	0.188
9	112.5	7,958	00 24 18	112.499	0.267
10	125.0	7,162	00 30 00	124.999	0.365
11	137.5	6,511	00 36 18	137.498	0.486
12	150.0	5,968	00 43 12	149.998	0.631
13	162.5	5,509	00 50 42	162.496	0.801
14	175.0	5,116	00 58 48	174.995	1.000
15	187.5	4,775	01 07 30	187.493	1.230
16	200.0	4,476	01 16 48	199.990	1.492
17	212.5	4,213	01 26 42	212.487	1.789
18	225.0	3,979	01 37 12	224.982	2.124
19	237.5	3,769	01 48 18	237.476	2.497
20	250.0	3,581	02 00 00	249.970	2.912
21	262.5	3,410	02 12 18	262.461	3.371
22	275.0	3,255	02 25 12	274.951	3.875
23	287.5	3,114	02 38 42	287.439	4.428
24	300.0	2,984	02 52 48	299.924	5.030
25	312.5	2,865	03 07 30	312.407	5.685
26	325.0	2,755	03 22 48	324.887	6.394
27	337.5	2,653	03 38 42	337.363	7.160
28	350.0	2,558	03 55 12	349.836	7.984
29	362.5	2,470	04 12 18	362.305	8.870
30	375.0	2,387	04 30 00	374.769	9.819
31	387.5	2,310	04 48 18	387.228	10.833
32	400.0	2,238	05 07 12	399.681	11.914

CURVED CHANNELS
SPIRAL CURVE TABLES
HYDRAULIC DESIGN CHART 660-2/2
(SHEET 1 OF 5)

No. 23 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 00 23	12.500	0.001
2	25.0	28,024	00 01 32	25.000	0.004
3	37.5	18,683	00 03 27	37.500	0.013
4	50.0	14,012	00 06 08	50.000	0.031
5	62.5	11,210	00 09 35	62.500	0.059
6	75.0	9,341	00 13 48	75.000	0.102
7	87.5	8,007	00 18 47	87.500	0.161
8	100.0	7,006	00 24 32	100.000	0.240
9	112.5	6,228	00 31 03	112.499	0.341
10	125.0	5,605	00 38 20	124.998	0.467
11	137.5	5,095	00 46 23	137.498	0.621
12	150.0	4,671	00 55 12	149.996	0.806
13	162.5	4,311	01 04 47	162.494	1.024
14	175.0	4,003	01 15 08	174.992	1.278
15	187.5	3,737	01 26 15	187.488	1.572
16	200.0	3,503	01 38 08	199.984	1.907
17	212.5	3,297	01 50 47	212.478	2.286
18	225.0	3,114	02 04 12	224.971	2.714
19	237.5	2,950	02 18 23	237.462	3.191
20	250.0	2,802	02 33 20	249.950	3.721
21	262.5	2,669	02 49 03	262.437	4.307
22	275.0	2,548	03 05 32	274.920	4.951
23	287.5	2,437	03 22 47	287.400	5.657
24	300.0	2,335	03 40 48	299.876	6.427
25	312.5	2,242	03 59 35	312.348	7.263
26	325.0	2,156	04 19 08	324.815	8.169
27	337.5	2,076	04 39 27	337.277	9.147
28	350.0	2,002	05 00 32	349.733	10.200
29	362.5	1,933	05 22 23	362.181	11.331
30	375.0	1,868	05 45 00	374.622	12.543
31	387.5	1,808	06 08 23	387.055	13.837
32	400.0	1,752	06 32 32	399.479	15.218

No. 28 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 00 28	12.500	0.001
2	25.0	23,020	00 01 52	25.000	0.005
3	37.5	15,346	00 04 12	37.500	0.016
4	50.0	11,510	00 07 28	50.000	0.037
5	62.5	9,208	00 11 40	62.500	0.072
6	75.0	7,673	00 16 48	75.000	0.124
7	87.5	6,577	00 22 52	87.500	0.196
8	100.0	5,755	00 29 52	99.999	0.292
9	112.5	5,115	00 37 43	112.499	0.415
10	125.0	4,604	00 46 40	124.998	0.568
11	137.5	4,185	00 56 28	137.496	0.756
12	150.0	3,837	01 07 12	149.994	0.981
13	162.5	3,541	01 18 52	162.491	1.246
14	175.0	3,289	01 31 28	174.988	1.556
15	187.5	3,069	01 45 00	187.483	1.913
16	200.0	2,877	01 59 28	199.976	2.321
17	212.5	2,708	02 14 52	212.467	2.783
18	225.0	2,558	02 31 12	224.956	3.303
19	237.5	2,423	02 48 28	237.443	3.884
20	250.0	2,302	03 06 40	249.926	4.530
21	262.5	2,192	03 25 48	262.406	5.243
22	275.0	2,093	03 45 52	274.881	6.027
23	287.5	2,002	04 06 52	287.352	6.886
24	300.0	1,918	04 28 48	299.817	7.822
25	312.5	1,842	04 51 40	312.275	8.840
26	325.0	1,771	05 15 28	324.726	9.943
27	337.5	1,705	05 40 12	337.169	11.133
28	350.0	1,644	06 05 52	349.604	12.414
29	362.5	1,588	06 32 28	362.028	13.790
30	375.0	1,535	07 00 00	374.440	15.264
31	387.5	1,485	07 28 28	386.841	16.839
32	400.0	1,439	07 57 52	399.228	18.518

No. 35 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 00 35	12.500	0.001
2	25.0	18,417	00 02 20	25.000	0.006
3	37.5	12,278	00 05 15	37.500	0.020
4	50.0	9,209	00 09 20	50.000	0.047
5	62.5	7,367	00 14 35	62.500	0.090
6	75.0	6,139	00 21 00	75.000	0.155
7	87.5	5,262	00 28 35	87.499	0.245
8	100.0	4,604	00 37 20	99.999	0.365
9	112.5	4,093	00 47 15	112.498	0.519
10	125.0	3,683	00 58 20	124.996	0.711
11	137.5	3,349	01 10 35	137.494	0.945
12	150.0	3,070	01 24 00	149.991	1.226
13	162.5	2,833	01 38 35	162.487	1.558
14	175.0	2,631	01 54 20	174.981	1.945
15	187.5	2,456	02 11 15	187.473	2.391
16	200.0	2,302	02 29 20	199.962	2.901
17	212.5	2,167	02 48 35	212.449	3.479
18	225.0	2,046	03 09 00	224.932	4.129
19	237.5	1,939	03 30 35	237.411	4.855
20	250.0	1,842	03 53 20	249.885	5.661
21	262.5	1,754	04 17 15	262.353	6.553
22	275.0	1,674	04 42 20	274.814	7.532
23	287.5	1,601	05 08 35	287.268	8.605
24	300.0	1,535	05 36 00	299.713	9.776
25	312.5	1,473	06 04 35	312.148	11.047
26	325.0	1,417	06 34 20	324.572	12.424
27	337.5	1,364	07 05 15	336.984	13.911
28	350.0	1,316	07 37 20	349.381	15.511
29	362.5	1,270	08 10 35	361.762	17.229
30	375.0	1,228	08 45 00	374.126	19.068
31	387.5	1,188	09 20 35	386.470	21.034
32	400.0	1,151	09 57 20	398.794	23.129

No. 44 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 00 44	12.500	0.001
2	25.0	14,650	00 02 56	25.000	0.008
3	37.5	9,767	00 06 36	37.500	0.025
4	50.0	7,325	00 11 44	50.000	0.059
5	62.5	5,860	00 18 20	62.500	0.113
6	75.0	4,883	00 26 24	75.000	0.195
7	87.5	4,186	00 35 56	87.499	0.308
8	100.0	3,662	00 46 56	99.998	0.459
9	112.5	3,256	00 59 24	112.497	0.652
10	125.0	2,930	01 13 20	124.994	0.893
11	137.5	2,664	01 28 44	137.491	1.188
12	150.0	2,442	01 45 36	149.986	1.541
13	162.5	2,254	02 03 56	162.479	1.958
14	175.0	2,093	02 23 44	174.969	2.445
15	187.5	1,953	02 45 00	187.457	3.006
16	200.0	1,831	03 07 44	199.940	3.647
17	212.5	1,724	03 31 56	212.419	4.373
18	225.0	1,628	03 57 36	224.892	5.190
19	237.5	1,542	04 24 44	237.359	6.102
20	250.0	1,465	04 53 20	249.818	7.116
21	262.5	1,395	05 23 24	262.268	8.236
22	275.0	1,332	05 54 56	274.707	9.467
23	287.5	1,274	06 27 56	287.134	10.815
24	300.0	1,221	07 02 24	299.547	12.285
25	312.5	1,172	07 38 20	311.945	13.881
26	325.0	1,127	08 15 44	324.324	15.610
27	337.5	1,085	08 54 36	336.684	17.477
28	350.0	1,046	09 34 56	349.022	19.485
29	362.5	1,010	10 16 44	361.334	21.641
30	375.0	977	11 00 00	373.619	23.948
31	387.5	945	11 44 44	385.874	26.413
32	400.0	916	12 30 56	398.095	29.040

CURVED CHANNELS
SPIRAL CURVE TABLES

HYDRAULIC DESIGN CHART 660-2/2
(SHEET 2 OF 5)

No. 56 Curve

n	L, ft	r, ft	Δs °	'	"	X, ft	Y, ft
0	0.0		00	00	00		0.0
1	12.5		00	00	56	12.500	0.002
2	25.0	11,510	00	03	44	25.000	0.010
3	37.5	7,674	00	08	24	37.500	0.032
4	50.0	5,755	00	14	56	50.000	0.075
5	62.5	4,604	00	23	20	62.500	0.144
6	75.0	3,837	00	33	36	74.999	0.248
7	87.5	3,289	00	45	44	87.498	0.392
8	100.0	2,878	00	59	44	99.997	0.584
9	112.5	2,558	01	15	36	112.495	0.830
10	125.0	2,302	01	33	20	124.991	1.137
11	137.5	2,093	01	52	56	137.485	1.512
12	150.0	1,918	02	14	24	149.977	1.961
13	162.5	1,771	02	37	44	162.466	2.492
14	175.0	1,644	03	02	56	174.950	3.111
15	187.5	1,535	03	30	00	187.430	3.825
16	200.0	1,439	03	58	56	199.903	4.641
17	212.5	1,354	04	29	44	212.369	5.565
18	225.0	1,279	05	02	24	224.826	6.604
19	237.5	1,212	05	36	56	237.272	7.765
20	250.0	1,151	06	13	20	249.705	9.054
21	262.5	1,096	06	51	36	262.124	10.477
22	275.0	1,046	07	31	44	274.525	12.043
23	287.5	1,001	08	13	44	286.907	13.756
24	300.0	959	08	57	36	299.267	15.624
25	312.5	921	09	43	20	311.601	17.653
26	325.0	885	10	30	56	323.906	19.849
27	337.5	853	11	20	24	336.179	22.219
28	350.0	822	12	11	44	348.417	24.768
29	362.5	794	13	04	56	360.614	27.503
30	375.0	767	14	00	00	372.766	30.430
31	387.5	743	14	56	56	384.869	33.554
32	400.0	719	15	55	44	396.918	36.882

No. 71 Curve

n	L, ft	r, ft	Δs °	'	"	X, ft	Y, ft
0	0.0		00	00	00		0.0
1	12.5		00	01	11	12.500	0.002
2	25.0	9,079	00	04	44	25.000	0.013
3	37.5	6,052	00	10	39	37.500	0.041
4	50.0	4,539	00	18	56	50.000	0.095
5	62.5	3,631	00	29	35	62.500	0.183
6	75.0	3,026	00	42	36	74.999	0.314
7	87.5	2,594	00	57	59	87.498	0.497
8	100.0	2,270	01	15	44	99.995	0.740
9	112.5	2,017	01	35	51	112.491	1.052
10	125.0	1,816	01	58	20	124.985	1.441
11	137.5	1,651	02	23	11	137.476	1.917
12	150.0	1,513	02	50	24	149.963	2.487
13	162.5	1,397	03	19	59	162.445	3.160
14	175.0	1,297	03	51	56	174.920	3.944
15	187.5	1,210	04	26	15	187.387	4.849
16	200.0	1,135	05	02	56	199.844	5.883
17	212.5	1,068	05	41	59	212.289	7.054
18	225.0	1,009	06	23	24	224.720	8.370
19	237.5	956	07	07	11	237.133	9.840
20	250.0	908	07	53	20	249.526	11.473
21	262.5	865	08	41	51	261.895	13.276
22	275.0	825	09	32	44	274.237	15.257
23	287.5	789	10	25	59	286.547	17.426
24	300.0	757	11	21	36	298.822	19.789
25	312.5	726	12	19	35	311.056	22.354
26	325.0	698	13	19	56	323.243	25.129
27	337.5	672	14	22	39	335.380	28.123
28	350.0	648	15	27	44	347.458	31.341
29	362.5	626	16	35	11	359.472	34.792
30	375.0	605	17	45	00	371.415	38.481
31	387.5	586	18	57	11	383.279	42.417

No. 90 Curve

n	L, ft	r, ft	Δs °	'	"	X, ft	Y, ft
0	0.0		00	00	00		0.0
1	12.5		00	01	30	12.500	0.003
2	25.0	7,162	00	06	00	25.000	0.016
3	37.5	4,775	00	13	30	37.500	0.052
4	50.0	3,581	00	24	00	50.000	0.[illegible]20
5	62.5	2,865	00	37	30	62.499	0.232
6	75.0	2,387	00	54	00	74.998	0.398
7	87.5	2,046	01	13	30	87.49[illegible]	0.630
8	100.0	1,790	01	36	00	99.992	0.938
9	112.5	1,592	02	01	30	112.486	1.333
10	125.0	1,432	02	30	00	124.976	1.827
11	137.5	1,302	03	01	30	137.462	2.429
12	150.0	1,194	03	36	00	149.941	3.152
13	162.5	1,102	04	13	30	162.411	4.005
14	175.0	1,023	04	54	00	174.872	4.999
15	187.5	955	05	37	30	187.319	6.[illegible]5
16	200.0	895	06	24	00	199.750	7.455
17	212.5	843	07	13	30	212.162	8.937
18	225.0	796	08	06	00	224.550	10.604
19	237.5	754	09	01	30	236.911	12.465
20	250.0	716	10	00	00	249.239	14.531
21	262.5	682	11	01	30	261.529	16.812
22	275.0	651	12	06	00	273.775	19.317
23	287.5	623	13	13	30	285.971	22.057
24	300.0	597	14	24	00	298.109	25.041
25	312.5	573	15	37	30	310.182	28.279
26	325.0	551	16	54	00	322.182	31.780
27	337.5	531	18	13	30	334.099	35.551
28	350.0	512	19	36	00	345.924	39.603

No. 113 Curve

n	L, ft	r, ft	Δs °	'	"	X, ft	Y, ft
0	0.0		00	00	00		0.0
1	12.5		00	01	53	12.500	0.003
2	25.0	5,704	00	07	32	25.000	0.021
3	37.5	3,803	00	16	57	37.500	0.065
4	50.0	2,852	00	30	08	50.000	0.151
5	62.5	2,282	00	47	05	62.499	0.291
6	75.0	1,901	01	07	48	74.997	0.500
7	87.5	1,630	01	32	17	87.494	0.791
8	100.0	1,426	02	00	32	99.988	1.178
9	112.5	1,268	02	32	33	112.478	1.674
10	125.0	1,141	03	08	20	124.962	2.294
11	137.5	1,037	03	47	53	137.439	3.050
12	150.0	951	04	31	12	149.906	3.956
13	162.5	878	05	18	17	162.360	5.027
14	175.0	815	06	09	08	174.798	6.274
15	187.5	761	07	03	45	187.215	7.713
16	200.0	713	08	02	08	199.606	9.355
17	212.5	671	09	04	17	211.967	11.214
18	225.0	634	10	10	12	224.291	13.303
19	237.5	600	11	19	53	236.571	15.639
20	250.0	570	12	33	20	248.801	18.222
21	262.5	543	13	50	33	260.970	21.076
22	275.0	519	15	11	32	273.071	24.209
23	287.5	496	16	36	17	285.092	27.633
24	300.0	475	18	04	48	297.024	31.359
25	312.5	456	19	37	05	308.853	35.397

CURVED CHANNELS
SPIRAL CURVE TABLES

HYDRAULIC DESIGN CHART 660-2/2
(SHEET 3 OF 5)

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

No. 139 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 02 19	12.500	0.004
2	25.0	4,637	00 09 16	25.000	0.025
3	37.5	3,092	00 20 51	37.500	0.080
4	50.0	2,319	00 37 04	49.999	0.185
5	62.5	1,855	00 57 55	62.498	0.358
6	75.0	1,546	01 23 24	74.996	0.615
7	87.5	1,325	01 53 31	87.490	0.973
8	100.0	1,159	02 28 16	99.981	1.449
9	112.5	1,031	03 07 39	112.466	2.059
10	125.0	927	03 51 40	124.943	2.821
11	137.5	843	04 40 19	137.408	3.751
12	150.0	773	05 33 36	149.858	4.866
13	162.5	713	06 31 31	162.289	6.181
14	175.0	662	07 34 04	174.694	7.715
15	187.5	618	08 41 15	187.068	9.482
16	200.0	580	09 53 04	199.404	11.499
17	212.5	546	11 09 31	211.694	13.782
18	225.0	515	12 30 36	223.928	16.345
19	237.5	488	13 56 19	236.096	19.205
20	250.0	464	15 26 40	248.187	22.375
21	262.5	442	17 01 39	260.188	25.869
22	275.0	422	18 41 16	272.086	29.702

No. 168 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 02 48	12.500	0.005
2	25.0	3,837	00 11 12	25.000	0.031
3	37.5	2,558	00 25 12	37.500	0.097
4	50.0	1,918	00 44 48	49.999	0.224
5	62.5	1,535	01 10 00	62.497	0.433
6	75.0	1,279	01 40 48	74.993	0.743
7	87.5	1,096	02 17 12	87.486	1.176
8	100.0	959	02 59 12	99.973	1.751
9	112.5	853	03 46 48	112.451	2.489
10	125.0	767	04 40 00	124.917	3.409
11	137.5	699	05 38 48	137.366	4.533
12	150.0	639	06 43 12	149.793	5.879
13	162.5	590	07 53 12	162.191	7.468
14	175.0	548	09 08 48	174.553	9.319
15	187.5	512	10 30 00	186.870	11.452
16	200.0	480	11 56 48	199.130	13.885
17	212.5	451	13 29 12	211.323	16.636
18	225.0	426	15 07 12	223.436	19.724
19	237.5	404	16 50 48	235.452	23.166
20	250.0	384	18 40 00	247.356	26.978

No. 200 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 03 20	12.500	0.006
2	25.0	3,223	00 13 20	25.000	0.036
3	37.5	2,149	00 30 00	37.500	0.115
4	50.0	1,611	00 53 20	49.999	0.267
5	62.5	1,289	01 23 20	62.496	0.515
6	75.0	1,074	02 00 00	74.991	0.885
7	87.5	921	02 43 20	87.480	1.400
8	100.0	806	03 33 20	99.961	2.084
9	112.5	716	04 30 00	112.430	2.962
10	125.0	645	05 33 20	124.882	4.058
11	137.5	586	06 43 20	137.310	5.394
12	150.0	537	08 00 00	149.707	6.996
13	162.5	496	09 23 20	162.063	8.885
14	175.0	460	10 53 20	174.367	11.086
15	187.5	430	12 30 00	186.607	13.619
16	200.0	403	14 13 20	198.769	16.508
17	212.5	379	16 03 20	210.834	19.772
18	225.0	358	18 00 00	222.786	23.432
19	237.5	339	20 03 20	234.602	27.507

No. 237 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 03 57	12.500	0.007
2	25.0	2,720	00 15 48	25.000	0.043
3	37.5	1,813	00 35 33	37.500	0.136
4	50.0	1,360	01 03 12	49.998	0.316
5	62.5	1,088	01 38 45	62.495	0.610
6	75.0	907	02 22 12	74.987	1.048
7	87.5	777	03 13 33	87.472	1.659
8	100.0	680	04 12 48	99.946	2.469
9	112.5	604	05 19 57	112.402	3.510
10	125.0	544	06 35 00	124.834	4.807
11	137.5	494	07 57 57	137.233	6.390
12	150.0	453	09 28 48	149.588	8.285
13	162.5	418	11 07 33	161.886	10.521
14	175.0	389	12 54 12	174.112	13.123
15	187.5	363	14 48 45	186.248	16.117
16	200.0	340	16 51 12	198.273	19.527
17	212.5	320	19 01 33	210.164	23.377

CURVED CHANNELS
SPIRAL CURVE TABLES

HYDRAULIC DESIGN CHART 660-2/2
(SHEET 4 OF 5)

No. 280 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 04 40	12.500	0.008
2	25.0	2,302	00 18 40	25.000	0.051
3	37.5	1,535	00 42 00	37.499	0.161
4	50.0	1,151	01 14 40	49.998	0.373
5	62.5	921	01 56 40	62.493	0.721
6	75.0	767	02 48 00	74.982	1.238
7	87.5	658	03 48 40	87.461	1.959
8	100.0	576	04 58 40	99.924	2.917
9	112.5	512	06 18 00	112.363	4.145
10	125.0	460	07 46 40	124.769	5.677
11	137.5	419	09 24 40	137.128	7.545
12	150.0	384	11 12 00	149.426	9.781
13	162.5	354	13 08 40	161.644	12.417
14	175.0	329	15 14 40	173.762	15.482
15	187.5	307	17 30 00	185.754	19.005
16	200.0	288	19 54 40	197.593	23.013

No. 340 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 05 40	12.500	0.010
2	25.0	1,896	00 22 40	25.000	0.062
3	37.5	1,264	00 51 00	37.499	0.196
4	50.0	948	01 30 40	49.996	0.453
5	62.5	758	02 21 40	62.489	0.876
6	75.0	632	03 24 00	74.973	1.504
7	87.5	542	04 37 40	87.442	2.379
8	100.0	474	06 02 40	99.888	3.541
9	112.5	421	07 39 00	112.298	5.031
10	125.0	379	09 26 40	124.659	6.889
11	137.5	345	11 25 40	136.952	9.153
12	150.0	316	13 36 00	149.154	11.862
13	162.5	292	15 57 40	161.239	15.050
14	175.0	271	18 30 40	173.177	18.754

No. 420 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 07 00	12.500	0.013
2	25.0	1,535	00 28 00	25.000	0.076
3	37.5	1,023	01 03 00	37.499	0.242
4	50.0	767	01 52 00	49.995	0.560
5	62.5	614	02 55 00	62.483	1.082
6	75.0	512	04 12 00	74.959	1.857
7	87.5	438	05 43 00	87.412	2.938
8	100.0	384	07 28 00	99.829	4.373
9	112.5	341	09 27 00	112.192	6.211
10	125.0	307	11 40 00	124.480	8.501
11	137.5	279	14 07 00	136.664	11.290
12	150.0	256	16 48 00	148.711	14.621
13	162.5	236	19 43 00	160.580	18.537

No. 520 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 08 40	12.500	0.016
2	25.0	1,240	00 34 40	25.000	0.095
3	37.5	826	01 18 00	37.498	0.299
4	50.0	620	02 18 40	49.992	0.693
5	62.5	496	03 36 40	62.475	1.339
6	75.0	413	05 12 00	74.937	2.299
7	87.5	354	07 04 40	87.365	3.636
8	100.0	310	09 14 40	99.738	5.410
9	112.5	275	11 42 00	112.029	7.682
10	125.0	248	14 26 40	124.204	10.509
11	137.5	225	17 28 40	136.220	13.946

No. 720 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 12 00	12.500	0.022
2	25.0	895	00 48 00	24.999	0.131
3	37.5	597	01 48 00	37.496	0.414
4	50.0	448	03 12 00	49.984	0.960
5	62.5	358	05 00 00	62.451	1.853
6	75.0	298	07 12 00	74.880	3.182
7	87.5	256	09 48 00	87.241	5.029
8	100.0	224	12 48 00	99.498	7.478
9	112.5	199	16 12 00	111.598	10.607
10	125.0	179	20 00 00	123.477	14.490

No 1080 Curve

n	L, ft	r, ft	Δs ° ' "	X, ft	Y, ft
0	0.0		00 00 00		0.0
1	12.5		00 18 00	12.500	0.033
2	25.0	597	01 12 00	24.999	0.196
3	37.5	398	02 42 00	37.491	0.622
4	50.0	298	04 48 00	49.964	1.439
5	62.5	239	07 30 00	62.391	2.778
6	75.0	199	10 48 00	74.730	4.766
7	87.5	171	14 42 00	86.919	[illegible]
8	100.0	149	19 12 00	98.873	11.167

CURVED CHANNELS
SPIRAL CURVE TABLES

HYDRAULIC DESIGN CHART 660-2/2
(SHEET 5 OF 5)

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 9-70

GIVEN:

Design $Q = 15{,}000$ cfs
Channel width $W = 50$ ft
Invert slope $S = 0.005$
Curve deflection angle $I = 45$ deg
Channel shape - rectangular
Design controls - Sheets 631 to 631-2, par 7b(2)

	Capacity	Curve geometry
Equivalent roughness k_s	0.007 ft	0.002 ft
Depth y	11.26 ft	10.33 ft
Velocity V	26.65 fps	29.05 fps
Critical depth d_c	14.0 ft	14.0 ft
Froude No.	1.40	1.59

REQUIRED:

Spiral (minimum length) and simple curve (minimum radius) geometries with invert banking

COMPUTE:

a. Simple curve radius (min)

$$r_{min} = \frac{4V^2W}{gy} = \frac{4(29.05)^2(50)}{(32.2)(10.33)} = 507.42 \text{ ft} \quad \text{(Eq 2, Sheet 660-1)}$$

b. Approximate banking (Chart 660-1) = $2\Delta y$

$$\frac{r}{W} = \frac{507.42}{50} = 10.14$$

For $V = 29.05$ fps and $\frac{r}{W} = 10.14$; $\frac{\Delta y}{C} = 2.6$

$\Delta y = 2.6(0.5) = 1.3$ ft

c. Spiral length (min) L

$L = 30\Delta y = 30(1.3) = 39$ ft (Eq 3)

d. Spiral curve geometry

For $r_{min} \approx 507$ and $L \approx 39$ use spiral curve No. 520 (Chart 660-2/2, Sheet 5 of 5)

$\Delta_s = 02°18'40''$

n	ΣL (ft)	r (ft)	δ_n* ° ' ''	X (ft)	Y (ft)
1	12.5		00 08 40	12.500	0.016
2	25.0	1,240	00 26 00	25.000	0.095
3	37.5	826	00 43 20	37.498	0.299
4	50.0	620	01 00 40	49.992	0.693
			02 18 40		

* $\delta_n = (2n - 1)\delta_1$ (Chart 660-2)

CHANNEL CURVE
EXAMPLE COMPUTATION

HYDRAULIC DESIGN CHART 660-2/3
(SHEET 1 OF 2)

e. Simple curve geometry (use $r = 620$ ft)

(1) Central angle θ (Chart 660-2)

$$\theta = I - 2\Delta_s = 45 - 2(02°18'40'')$$
$$= 45 - (04°37'20'') = 40°22'40''$$

(2) Curve length L_c (Chart 660-2)

$$L_c = \frac{(I - 2\Delta_s)r}{57.2958} = \frac{(40°22'40'')(620)}{57.2958}$$
$$= \frac{40.38(620)}{57.2958} = 436.95 \text{ ft}$$

f. Total curve length L_c

$$L_T = 2L + L_c = 2(50) + 436.95 = 536.95 \text{ ft}$$

g. Corrected invert banking $= 2\Delta y$

$$\frac{r}{W} = \frac{620}{50} = 12.40$$

For $V = 29.05$ fps and $\frac{r}{W} = 12.40$

$$\frac{\Delta y}{C} = 2.2 \quad \text{(Chart 660-1)}$$

$$\Delta y = 2.2C = 2.2(0.5) = 1.10 \text{ ft}$$
$$2\Delta y = 2.20 \text{ ft}$$

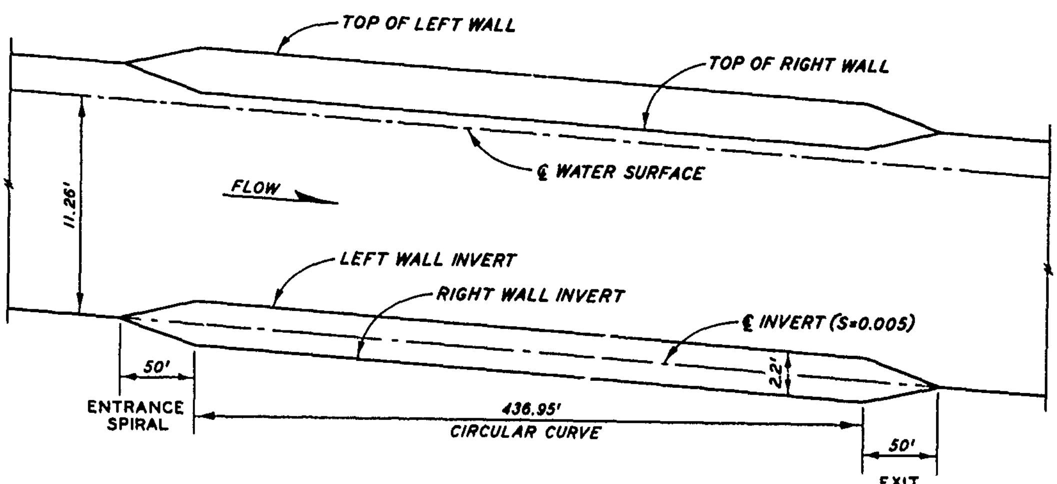
h. Maximum allowable Δy_{max}

$$2\Delta y_{max} = 0.18W = 0.18(50) = 9.0 \text{ ft} \quad \text{(Eq 3, Sheet 660-1)}$$
$$\Delta y_{max} = 4.5 \text{ ft} > \Delta y = 1.10 \text{ ft} \quad \text{(item g) OK}$$

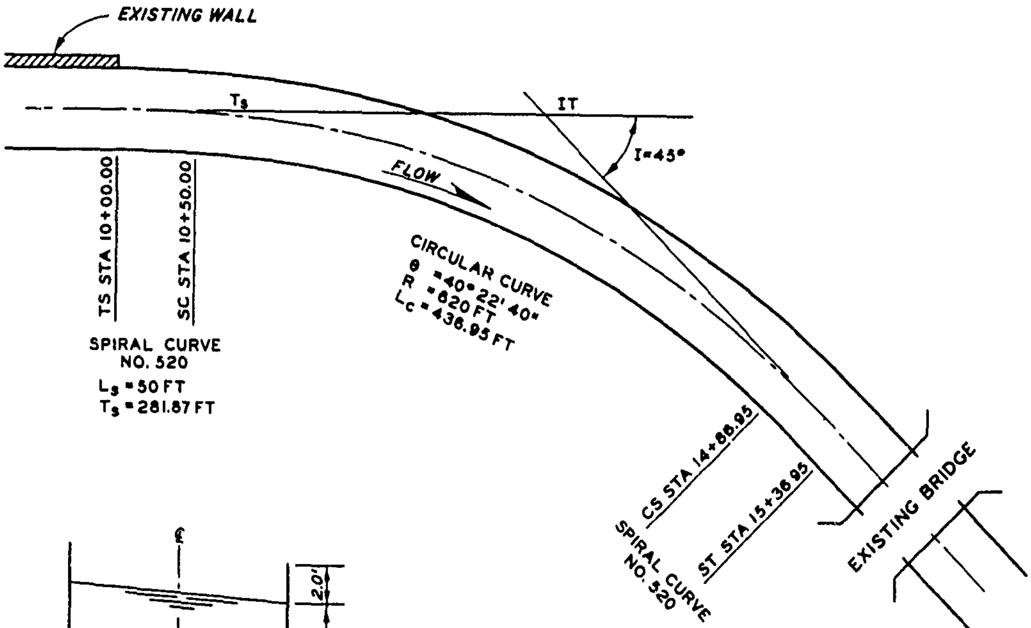
i. Curve tangent distance T_s

$$T_s = X - r \sin \Delta_s + (Y + r \cos \Delta_s) \tan \frac{I}{2}$$

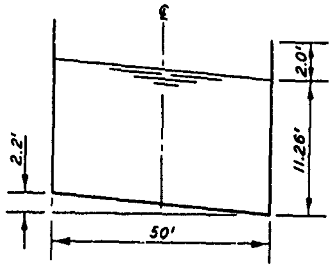
$$49.992 - 620 \sin(02°18'40'') + (0.693 + 620 \cos 02°18'40'') \tan 22°30'00''$$
$$49.992 - 620(0.04033) + \left[0.693 + 620(0.99919)\right] 0.41421$$
$$49.992 - 25.005 + (0.693 + 619.498)0.41421$$
$$24.987 + (620.191)0.41421 = 281.87 \text{ ft}$$



PROFILE



PLAN



CURVE SECTION

HYDRAULIC ELEMENTS – K=0.007 FT

STA TO STA		SECTION	SLOPE	Y_c, FT	V, FPS	Y, FT	F	Q, CFS
10+00	18+00	50' RECT	0.005	14.0	26.65	11.26	1.4	15,000

CURVED CHANNELS

EXAMPLE PLAN AND PROFILE

HYDRAULIC DESIGN CHART 660-2/4

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 9-70

HYDRAULIC DESIGN CRITERIA

SHEET 703-1

RIPRAP PROTECTION

TRAPEZOIDAL CHANNEL, 60 DEG-BEND

BOUNDARY SHEAR DISTRIBUTION

1. Riprap used to aid in the stabilization of natural streams and artificial channels is most commonly placed in the vicinity of bends. Procedures for estimating the required size of riprap in straight channels have been presented by the U. S. Army Engineer Waterways Experiment Station[1] and Office, Chief of Engineers.[2] No similar procedure has been developed for evaluating riprap size for channel bends. Hydraulic Design Chart 703-1 is based on laboratory tests at the Massachusetts Institute of Technology (MIT)[3] and should be useful for estimating relative boundary shear distribution in simple channel bends having trapezoidal cross sections, moderate side slopes, and approximately 60-deg deflection angles. It may also serve as a general guide for riprap gradation in natural channel bends of similar geometry. Shear distribution diagrams for other bend geometries and flow conditions have been published.[3,4]

2. Laboratory studies of boundary shear in open channel bends of trapezoidal cross section[3,5] indicate that the highest boundary shear caused by the bend geometry occurs immediately downstream from the bend and along the outside bank. Another area of high boundary shear is located at the inside of the bend. The relative boundary shear distribution in a simple bend with a rough boundary is given in Chart 703-1. The chart is based on fig. 21 of the MIT report.[3]

3. Experimental Data. Laboratory tests on smooth channel bends have been made at MIT,[3] at U. S. Bureau of Reclamation,[5] and at the University of Iowa.[6] In addition, limited tests on rough channel bends have been made at MIT. In the latter tests, the channel was roughened by fixing 0.18- by 0.10- by 0.10-in. parallelepipeds to the boundary in a random manner which resulted in an absolute roughness height of 0.10 in. The MIT test channel was 24 in. wide with 1 on 2 side slopes. The boundary shear distribution pattern has been generally found to be the same in all tests on simple curves having smooth and rough boundary conditions. However, the magnitude of the ratio of bend local boundary shear to the average boundary shear in the approach channel appears to be a function of the channel and bend geometry. Some work has also been done at MIT[3] on boundary shear distribution in double and reverse curve channels.

4. Application. Extensive variation in riprap gradation throughout a bend may not be practical or economical. However, increasing the 50 percent rock size and the thickness of the riprap blanket in areas of expected high boundary shear is recommended. Chart 703-1 can be used as a

guide for defining the location and extent of these areas in simple channel bends. The boundary shear ratios should be less than those shown in Chart 703-1 for bends with smaller deflection angles or with larger ratios of bend radius to water-surface width (r/w).

5. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Hydraulic Design of Rock Riprap, by F. B. Campbell. Miscellaneous Paper No. 2-777, Vicksburg, Miss., February 1966.

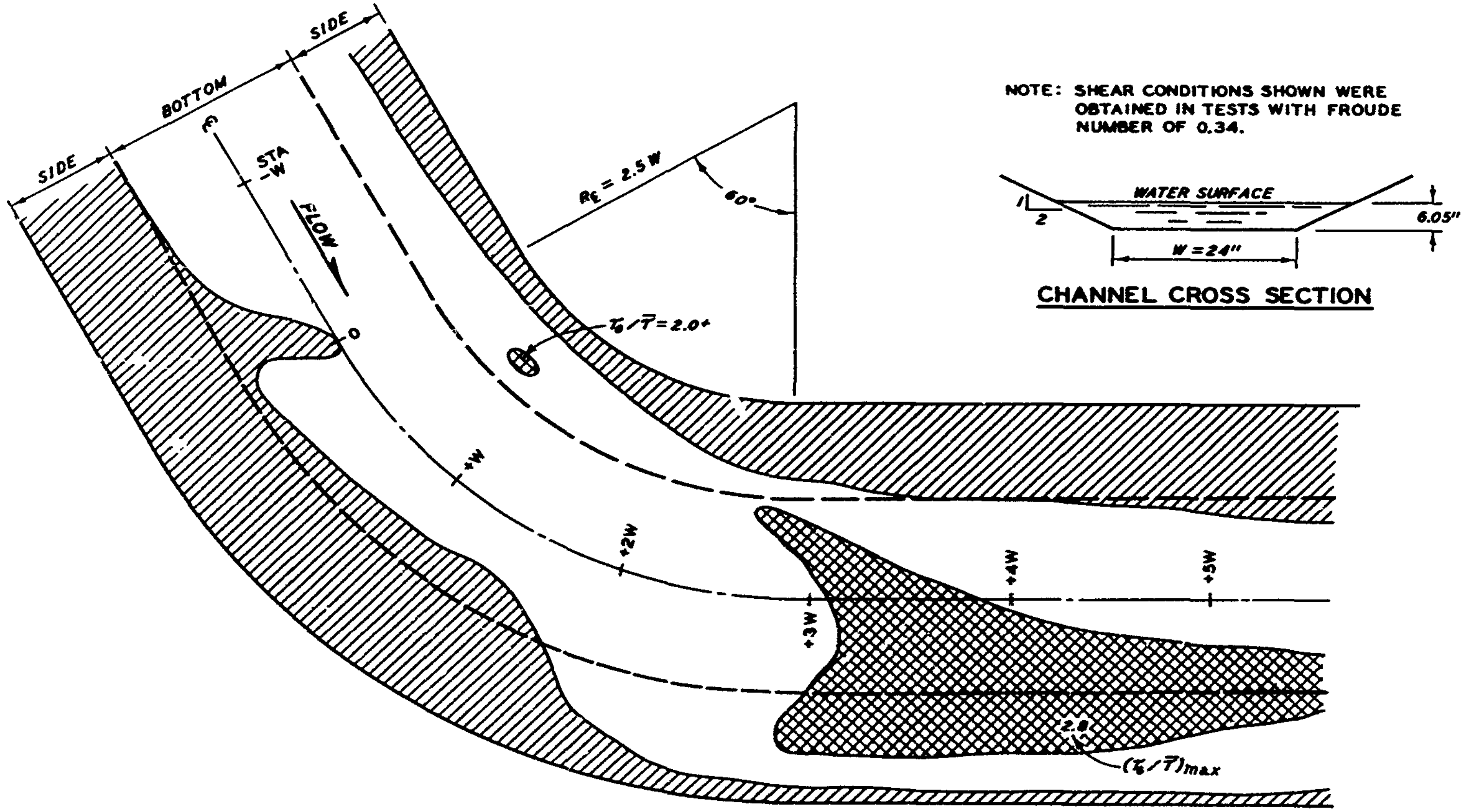
(2) U. S. Army Engineer, Office, Chief of Engineers, Stone Riprap Protection for Channels, by S. B. Powell.

(3) Ippen, A. T., and others, Stream Dynamics and Boundary Shear Distributions for Curved Trapezoidal Channels. Report No. 47, Hydrodynamics Laboratory, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, January 1962.

(4) Ippen, A. T., and Drinker, P. A., "Boundary shear stress in trapezoidal channels." ASCE, Hydraulics Division, Journal, vol 88, HY 5, paper 3273 (September 1962), pp 143-179.

(5) U. S. Bureau of Reclamation, Progress Report No. 1--Boundary Shear Distribution Around a Curve in a Laboratory Canal, by E. R. Zeigler. Hydraulics Branch Report No. HYD 526, 26 June 1964.

(6) Yen, Ben-Chie, Characteristics of Subcritical Flow in a Meandering Channel. Institute of Hydraulic Research, University of Iowa, Iowa City, 1965.



NOTE: SHEAR CONDITIONS SHOWN WERE OBTAINED IN TESTS WITH FROUDE NUMBER OF 0.34.

CHANNEL CROSS SECTION

LEGEND

- (diagonal hatching) $\tau_0 < \bar{\tau}$
- (blank) $\bar{\tau} < \tau_0 < 2.0\bar{\tau}$
- (cross-hatching) $2.0\bar{\tau} < \tau_0 < 2.8\bar{\tau}$

τ_0 = LOCAL BOUNDARY SHEAR STRESS

$\bar{\tau}$ = AVERAGE BOUNDARY SHEAR STRESS IN APPROACH FLOW.

TRAPEZOIDAL CHANNEL 60-DEG BEND

BOUNDARY SHEAR DISTRIBUTION

HYDRAULIC DESIGN CHART 703-1

WES 1-68

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

HYDRAULIC DESIGN CRITERIA

SHEET 704

ICE THRUST ON HYDRAULIC STRUCTURES

1. The expansion of an ice sheet as the result of a rise in air temperature can develop large thrusts against adjacent structures. The magnitude of this thrust is dependent upon the thickness of the ice sheet, the rate of air temperature rise, the amount of lateral restraint, and the extent of direct penetration of solar energy. Ice pressures from 3350 to 30,000 lb per lin ft[1] have been used for design purposes. EM 1110-2-2200[3] suggests a unit pressure of not more than 5000 lb per sq ft of contact area and indicates that ice thickness in the United States will not normally exceed 2 ft.

2. Although the work of Rose[2] stimulated a number of studies on ice pressure, the graphs proposed by him are of value for design purposes. These graphs are reproduced in HDC 704.

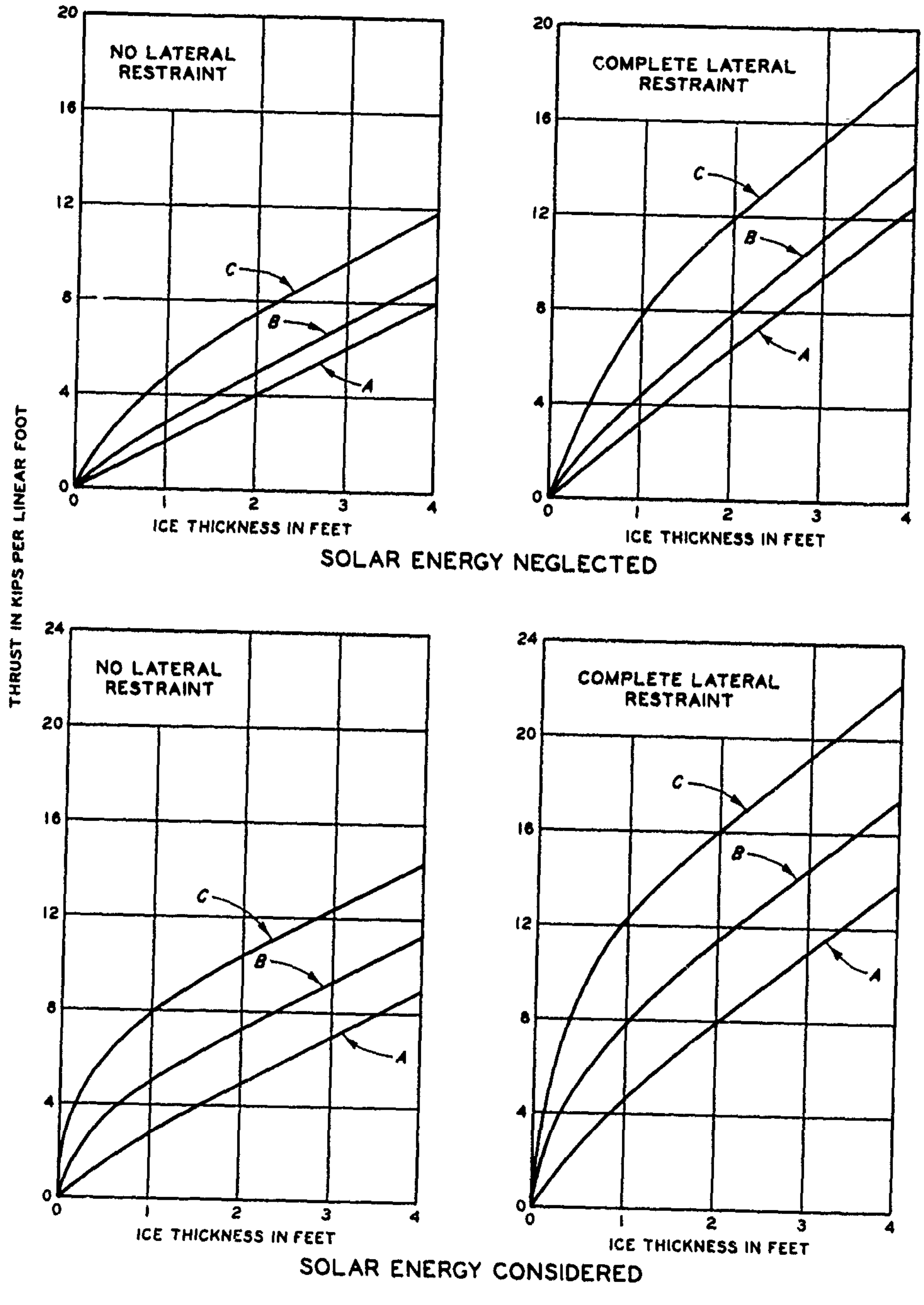
3. The ice thrust curves in HDC 704 are for ice thicknesses up to 4 ft and hourly air temperature rises of 5, 10, and 15°F. Separate curves are presented to show the effects of lateral restraint and solar radiation. The expected ice thicknesses, air temperature rise, and possible snow blanket thickness are dependent upon geographical location and elevation above sea level. In the region of chinook winds rapid air temperature rises can occur. The U. S. Weather Bureau has recorded a 49°F rise in two minutes at Spearfish, S. Dak. When the ice sheet is confined by steep banks close to the structure, spillway piers, or other vertical restrictions, the criteria for complete lateral restraint should be used. The direct effects of solar energy on the thrust are eliminated when the ice sheet is insulated by a blanket of snow only a few inches thick.

4. References.

(1) American Society of Civil Engineers, "Ice pressure against dams: A symposium." Transactions, American Society of Civil Engineers, vol 119 (1954), pp 1-42.

(2) Rose, E., "Thrust exerted by expanding ice sheet." Transactions, American Society of Civil Engineers, vol 112 (1947), pp 871-900.

(3) U. S. Army, Office, Chief of Engineers, Engineering and Design, Gravity Dam Design. EM 1110-2-2200, 25 September 1958.



LEGEND

A - AIR TEMPERATURE RISE OF 5°F PER HOUR
B - AIR TEMPERATURE RISE OF 10°F PER HOUR
C - AIR TEMPERATURE RISE OF 15°F PER HOUR

NOTE: CURVES BY ROSE, TRANSACTIONS, ASCE, VOL 112, 1947.

ICE THRUSTS ON HYDRAULIC STRUCTURES

HYDRAULIC DESIGN CHART 704

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 10-61

HYDRAULIC DESIGN CRITERIA

SHEET 711

LOW-MONOLITH DIVERSION

DISCHARGE COEFFICIENTS

1. Purpose. Several monoliths of the spillway section of a concrete gravity dam are occasionally left at a low elevation during spillway construction for diversion of floodflows. Information on the discharge characteristics of these monoliths is necessary for determining the number of monoliths required to allow floodflows to pass safely. HDC 711 should serve as a guide for selection of discharge coefficients for this purpose.

2. Free Overflow. The flow over low concrete monoliths is generally treated as flow over a broad-crested weir. The equation for free discharge is:

$$Q = C_f \ (L - 2\ KH)\ H^{3/2}$$

where C_f is an empirical coefficient, L is the length of opening transverse to the flow, H is the head on the weir, and K is an end contraction coefficient. The value of K is conventionally taken to be 0.10 for square-end contractions. The free-flow coefficient C_f varies with the ratio of head H to width B of the broad-crested weir in the direction of flow. HDC 711a shows the variation of C_f with H/B resulting from investigations summarized by Tracy.[1] Kindsvater[2] has recently shown the effect of boundary layer development on broad-crested-weir discharge. The rate of development is a function of the bottom roughness. However, present knowledge of this effect does not justify considering boundary layer development for diversion flow computations. The curve resulting from the classical experiments of Bazin[3] as shown by the solid curve in HDC 711a is recommended for general design purposes.

3. Submergence Effect. Discharge coefficients for broad-crested weirs are not usually affected until the depth of submergence is about 0.67 or more of the head on the weir. The phenomenon is commonly expressed in terms of the ratio of the coefficient of the submerged weir to that of the unsubmerged weir C_s/C_f as a function of the ratio of the tailwater depth on the weir to the head on the weir H_2/H_1. Available data indicate that sharp-crested-weir coefficients are more sensitive to submergence than broad-crested-weir coefficients.

4. Available data on the effects of submergence on discharge coefficients for both sharp- and broad-crested weirs[2,4,5,6] are summarized in HDC 711b. As far as is known, rectangular broad-crested weirs have not been subjected to submergence tests. A suggested design curve for submerged low monoliths is given in the chart.

5. Application. The suggested design curves given in HDC 711 should serve as guides for estimating diversion flows over low monoliths. In cases where the head-discharge relation may be critical, a more exact relation should be obtained by hydraulic model investigation. A model study of proposed low-monolith diversion schemes for Allatoona Dam[7] was made because of critical diversion requirements.

6. References.

(1) Tracy, H. J., Discharge Characteristics of Broad-Crested Weirs. U. S. Geological Survey Circular 397, 1957.

(2) Kindsvater, C. E., Discharge Characteristics of Embankment-Shaped Weir; Studies of Flow of Water Over Weirs and Dams. U. S. Geological Survey Water-Supply Paper 1617-A, 1964.

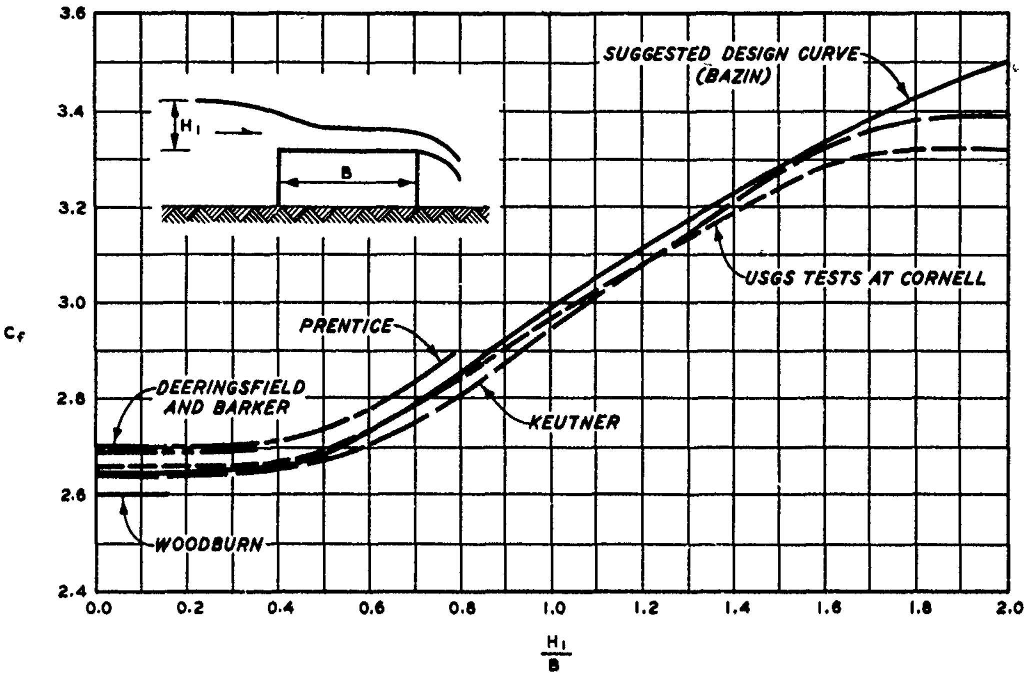
(3) Bazin, M. H., "Experiences nouvelles sur l'ecoulement en diversoir." Annales des Ponts et Chaussées, vol 7, Series 7, 1896.

(4) U. S. Geological Survey, Weir Experiments, Coefficients, and Formulas, by R. E. Horton. Water-Supply Paper No. 200, 1907, p 146.

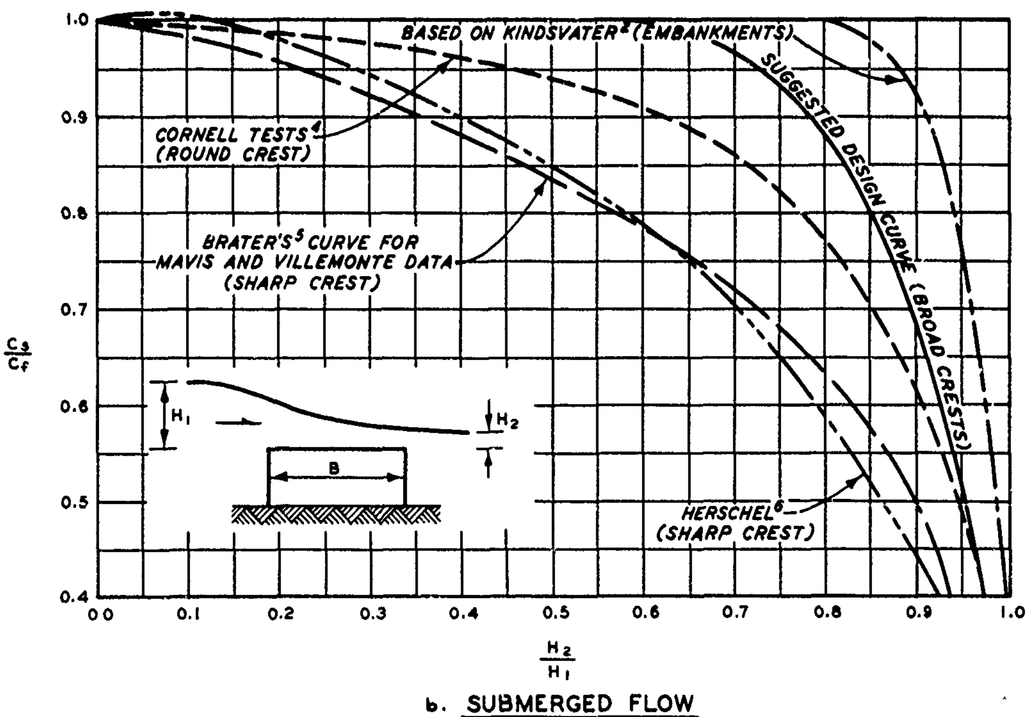
(5) King, H. W., Handbook of Hydraulics for the Solution of Hydraulic Problems, revised by E. F. Brater, 4th ed. McGraw-Hill Book Co., Inc., New York, N. Y., 1954, pp 4-18.

(6) King, H. W., Handbook of Hydraulics for the Solution of Hydraulic Problems, 3d ed. McGraw-Hill Book Co., Inc., New York, N. Y., 1939, p 99.

(7) U. S. Army Engineer Waterways Experiment Station, CE, Sluices and Diversion Scheme for Allatoona Dam, Etowah River, Georgia; Model Investigation. Technical Memorandum No. 214-2, Vicksburg, Miss., November 1948.



a. FREE FLOW



b. SUBMERGED FLOW

NOTE: C_f = FREE-FLOW COEFFICIENT
C_s = SUBMERGED-FLOW COEFFICIENT
NEGLIGIBLE VELOCITY OF APPROACH
RAISED NUMBERS ON SUBMERGED FLOW CHART ARE REFERENCE NUMBERS FROM TEXT.

LOW-MONOLITH DIVERSION
DISCHARGE COEFFICIENTS
HYDRAULIC DESIGN CHART 711

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI
WES 1-66

SHEET 712-1

STONE STABILITY

VELOCITY VS STONE DIAMETER

1. Purpose. Hydraulic Design Chart 712-1 can be used as a guide for the selection of rock sizes for riprap for channel bottom and side slopes downstream from stilling basins and for rock sizes for river closures. Recommended stone gradation for stilling basin riprap is given in paragraph 6.

2. Background. In 1885 Wilfred Airy[1] showed that the capacity of a stream to move material along its bed by sliding is a function of the sixth power of the velocity of the water.[1] Henry Law applied this concept to the overturning of a cube,[2] and in 1896 Hooker[2] illustrated its application to spheres. In 1932 and 1936 Isbash published coefficients for the stability of rounded stones dropped in flowing water.[3,4] The design curves given in Chart 712-1 have been computed using Airy's law and the experimental coefficients for rounded stones published by Isbash.

3. Theory. According to Isbash the basic equation for the movement of stone in flowing water can be written as:

$$V = C \left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2} (D)^{1/2} \tag{1}$$

where

V = velocity, fps

C = a coefficient

g = acceleration of gravity, ft/sec^2

γ_s = specific weight of stone, lb/ft^3

γ_w = specific weight of water, lb/ft^3

D = stone diameter, ft

The diameter of a spherical stone in terms of its weight W is

$$D = \left(\frac{6W}{\pi\gamma_s}\right)^{1/3} \tag{2}$$

Substituting for D in equation 1 results in

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}\left(\frac{6W}{\pi\gamma_s}\right)^{1/6}$$

which describes Airy's law stated in paragraph 2.

4. Experimental Results. Experimental data on stone movement in flowing water from the early (1786) work of DuBuat[5] to the more recent Bonneville Hydraulic Laboratory tests[6] have been shown to confirm Airy's law and Isbash's stability coefficients.[7] The published experimental data are generally defined in terms of bottom velocities. However, some are in terms of average flow velocities and some are not specified. The Isbash coefficients are from tests with essentially no boundary layer development and the average flow velocities are representative of the velocity against stone. When the stone movement resulted by sliding, a coefficient of 0.86 was obtained. When movement was effected by rolling or overturning, a coefficient of 1.20 resulted. Extensive U. S. Army Engineer Waterways Experiment Station laboratory testing for the design of riprap below stilling basins indicates that the coefficient of 0.86 should be used with the average flow velocity over the end sill for sizing stilling basin riprap because of the excessively high turbulence level in the flow. For impact-type stilling basins, the Bureau of Reclamation[8] has adopted a riprap design curve based on field and laboratory experience and on a study by Mavis and Laushey.[9] The Bureau curve specifies rock weighing 165 lb/ft^3 and is very close to the Isbash curve for similar rock using a stability coefficient of 0.86.

5. Application. The curves given in Chart 712-1 are applicable to specific stone weights of 135 to 205 lb/ft^3. The use of the average flow velocity is desirable for conservative design. The solid-line curves are recommended for stilling basin riprap design and other high-level turbulence conditions. The dashed line curves are recommended for river closures and similar low-level turbulence conditions. Riprap bank and bed protection in natural and artificial flood-control channels should be designed in accordance with reference 10.

6. Stilling Basin Riprap.

a. Size. The W_{50} stone weight and the D_{50} stone diameter for establishing riprap size for stilling basins can be obtained using Chart 712-1 in the manner indicated by the heavy arrows thereon. The effect of specific weight of the rock on the required size is indicated by the vertical spread of the solid line curves.

b. Gradation. The following size criteria should serve as guidelines for stilling basin riprap gradation.

(1) The lower limit of W_{50} stone should not be less than the weight of stone determined using the appropriate "Stilling Basins" curve in Chart 712-1.

(2) The upper limit of W_{50} stone should not exceed the weight that can be obtained economically from the quarry or the size that will satisfy layer thickness requirements as specified in paragraph 6c.

(3) The lower limit of W_{100} stone should not be less than two times the lower limit of W_{50} stone.

(4) The upper limit of W_{100} stone should not be more than five times the lower limit of W_{50} stone, nor exceed the size that can be obtained economically from the quarry, nor exceed the size that will satisfy layer thickness requirements as specified in paragraph 6c.

(5) The lower limit of W_{15} stone should not be less than one-sixteenth the upper limit of W_{100} stone.

(6) The upper limit of W_{15} stone should be less than the upper limit of W_{50} stone as required to satisfy criteria for graded stone filters specified in EM 1110-2-1901.

(7) The bulk volume of stone lighter than the W_{15} stone should not exceed the volume of voids in the revetment without this lighter stone.

(8) W_0 to W_{25} stone may be used instead of W_{15} stone in criteria (5), (6), and (7) if desirable to better utilize available stone sizes.

c. Thickness. The thickness of the riprap protection should be $2D_{50\ max}$ or $1.5D_{100\ max}$, whichever results in the greater thickness.

d. Extent. Riprap protection should extend downstream to where nonerosive channel velocities are established and should be placed sufficiently high on the adjacent bank to provide protection from wave wash during maximum discharge. The required riprap thickness is determined by substituting values for these relations in equation 2.

7. References.

(1) Shelford, W., "On rivers flowing into tideless seas, illustrated by the river Tiber." Proceedings, Institute of Civil Engineers, vol 82 (1885).

(2) Hooker, E. H., "The suspension of solids in flowing water." Transactions, American Society of Civil Engineers, vol 36 (1896), pp 239-340.

(3) Isbash, S. V., Construction of Dams by Dumping Stones in Flowing

Water, Leningrad, 1932. Translated by A. Dorijikov, U. S. Army Engineer District, Eastport, CE, Maine, 1935.

(4) ________, "Construction of dams by depositing rock in running water." Transactions, Second Congress on Large Dams, vol 5 (1936), pp 123-136.

(5) DuBuat, P. L. G., Traite d'Hydraulique. Paris, France, 1786.

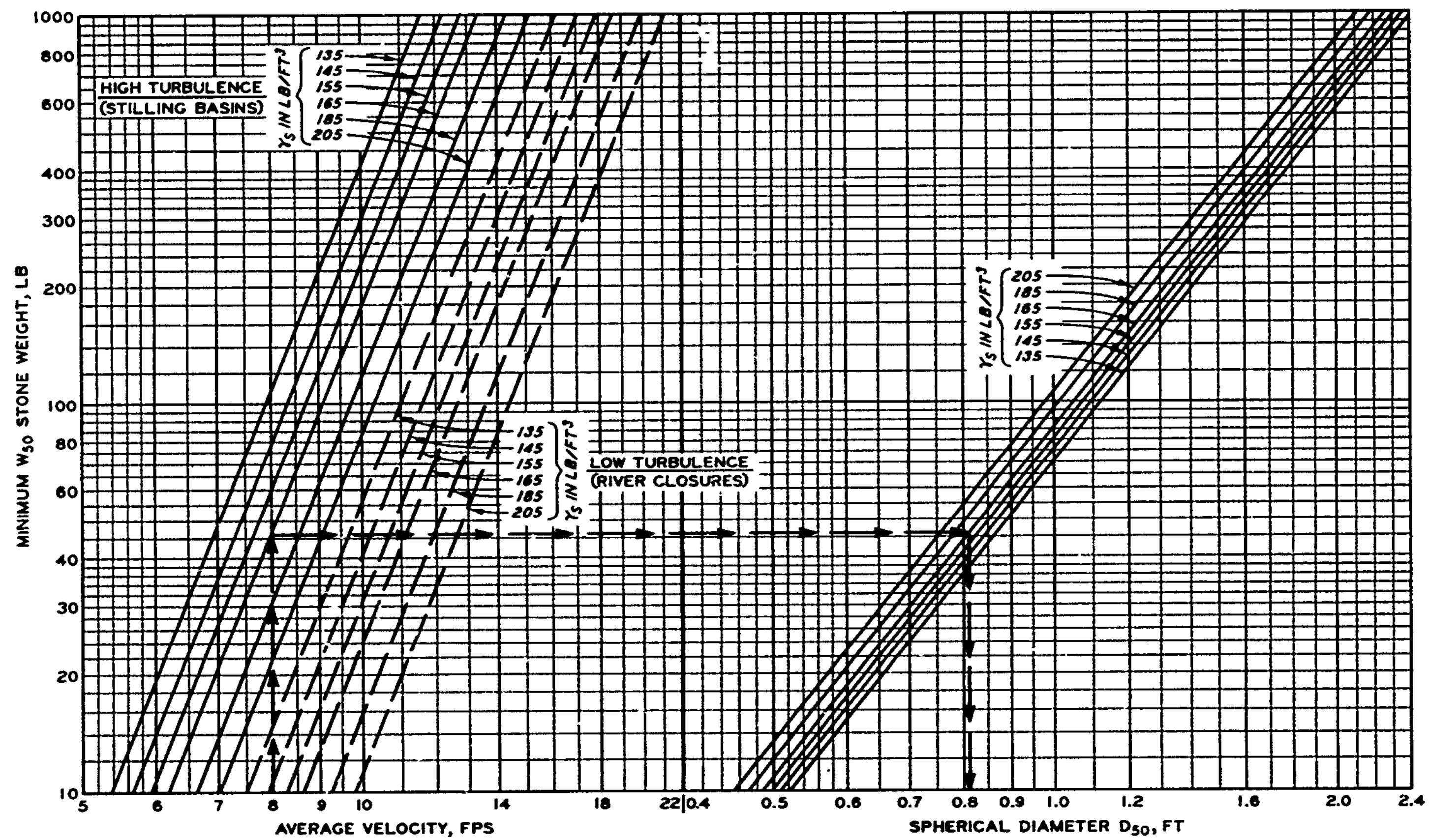
(6) U. S. Army Engineer District, Portland, CE, McNary Dam - Second Step Cofferdam Closure. Bonneville Hydraulic Laboratory Report No. 51-1, 1956.

(7) U. S. Army Engineer Waterways Experiment Station, CE, Velocity Forces on Submerged Rocks. Miscellaneous Paper No. 2-265, Vicksburg, Miss., April 1958.

(8) U. S. Bureau of Reclamation, Stilling Basin Performance; An Aid in Determining Riprap Sizes, by A. J. Peterka. Hydraulic Laboratory Report No. HYD-409, Denver, Colo., 1956.

(9) Mavis, F. T. and Laushey, L. M., "A reappraisal of the beginning of bed movement - competent velocity." Second Meeting, International Association for Hydraulic Structure Research, Stockholm, Sweden, 1948. See also Civil Engineering, vol 19 (January 1949), pp 38, 39, and 72.

(10) U. S. Army, Office, Chief of Engineers, Engineering and Design; Hydraulic Design of Flood Control Channels. EM 1110-2-1601, Washington, D. C., 1 July 1970.



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_S-\gamma_W}{\gamma_W}\right)\right]^{1/2}(D_{50})^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_S}\right)^{1/3}$$

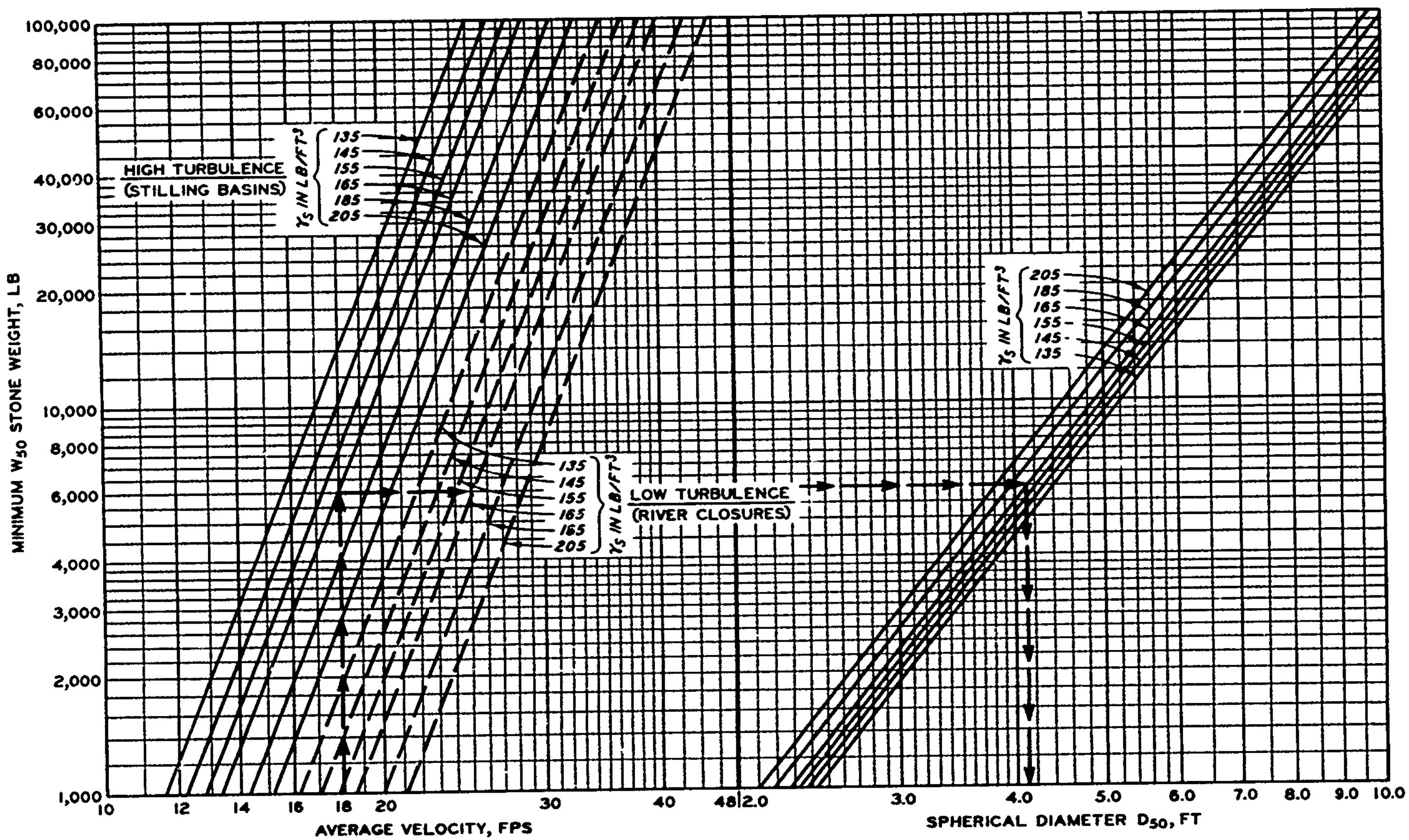
WHERE:
- V = VELOCITY, FPS
- γ_S = SPECIFIC STONE WEIGHT, LB/FT^3
- γ_W = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)

REV 8-58, 9-70 WES 6-57

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_S-\gamma_W}{\gamma_W}\right)\right]^{1/2}\left(D_{50}\right)^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_S}\right)^{1/3}$$

WHERE:
- V = VELOCITY, FPS
- γ_S = SPECIFIC STONE WEIGHT, LB/FT^3
- γ_W = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT^3
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC^2

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 2 OF 2)

REV 8-58, 9-70 WES 6-57

PREPARED BY U S ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG MISSISSIPPI

HYDRAULIC DESIGN CRITERIA

SHEETS 722-1 TO 722-3

STORM DRAIN OUTLETS

FIXED ENERGY DISSIPATORS

1. Purpose. Storm drains frequently terminate in unstable channels and gullies. Under these conditions dissipation of the energy of the outflow is required to prevent serious erosion and potential undermining and subsequent failure of the storm drains. Adequate energy dissipation can be accomplished by extensive riprap protection[1,2] or by construction of specially designed fixed energy dissipators.[3,4,5,6]

2. Hydraulic Design Charts (HDC's) 722-1 to -3 present design criteria for three types of laboratory tested energy dissipators.[3] Each type has its advantages and limitations. Selection of the optimum type and size is dependent upon local tailwater conditions, maximum expected discharge, and economic considerations.

3. Stilling Wells. The stilling well energy dissipator shown in HDC 722-1 was developed at the U. S. Army Engineer Waterways Experiment Station (WES).[3] Energy dissipation in this stilling well is relatively independent of tailwater and is accomplished by flow expansion in the well, by impact of the fluid on the base and wall of the well, and by the change in momentum resulting from redirection of the flow to vertically upward. WES laboratory tests[3] indicated that the structure performs satisfactorily for flow-pipe diameter ratios ($Q/D_o^{2.5}$) up to 10 with a well-pipe diameter ratio of 5.

4. HDC 722-1 shows the relation between storm drain diameter, well diameter, and discharge. Designing for operation beyond the limits shown in HDC 722-1 is not recommended. Intermediate ratios of stilling well-drain pipe diameters within the limits shown in HDC 722-1 can be computed using the equation given in this chart.

5. Impact Energy Dissipators. The U. S. Bureau of Reclamation (USBR)[5] has developed an impact energy dissipator which is an effective stilling device even with deficient tailwater. The dimensions of this energy dissipator in terms of its width are shown in HDC 722-2. Energy dissipation in the basin is accomplished by the impact of the entering jet on the vertically hanging baffle and by the eddies that are formed following impact on the baffle.

6. HDC 722-2 shows the relation between storm drain diameters, basin width, and discharge. WES laboratory tests[3] showed that this structure properly designed performs satisfactorily for $Q/D_o^{2.5}$ ratios up to 21. Intermediate ratios of basin widths within the limits shown in HDC 722-2 can be computed using the equation given in this chart. Design for operation beyond these limits is not recommended. The WES

tests also showed that optimum energy dissipation for the design flow occurs with the tailwater midway up the hanging baffle. Excessive tailwater should be avoided as this causes flow over the top of the baffle.

7. Hydraulic Jump Energy Dissipators. The St. Anthony Falls Hydraulic Laboratory (SAFHL)[6] has developed the hydraulic jump energy dissipator shown in HDC 722-3. Design equations for dimensionalizing the structure in terms of the square of the Froude number of the flow entering the dissipator are also given in the chart. WES laboratory tests[3] showed that this type of stilling basin performs satisfactorily for ratios of $Q/D_o^{2.5}$ up to 9.5 with a basin width three times the storm drain diameter. WES tests were limited to basin widths of 1, 2, and 3 times the drain diameter with drops (drain invert to stilling basin) of 0.5 and 2 times the drain diameter. Parallel stilling basin walls were used for basin width-drain diameter ratios of 1 and 2. The transition wall flare was continued through the basin for $W = 3D_o$. Parallel basin sidewalls are generally recommended for best performance. Transition sidewall flare (1:D') during the WES tests was fixed at 1 on 8. The invert transition to the stilling basin should conform to the geometry of the trajectory of a flow not less than 1.25 times the drain outlet portal design velocity.

8. HDC 722-3 shows the relation between storm drain diameter and discharge for stilling basin widths up to 3 times the drain diameter which results in satisfactory performance. WES tests have been restricted to the limits shown in HDC 722-3, and the equation given in the chart can be used to compute intermediate basin width-drain diameter ratios within those limits. General WES model tests of outlet works indicate that this equation also applies to ratios greater than the maximum shown in the chart. However, outlet portal velocities exceeding 60 fps are not recommended for designs containing chute blocks. This chart does not reflect the outlet invert transition effects on basin performance. The design of the basin itself (HDC 722-3) is dependent upon the depth and velocity of the flow as it enters the basin. The values should be computed taking into account the drain outlet transition geometry.

9. Riprap Protection. Riprap protection in the immediate vicinity of the energy dissipator is recommended. Preliminary, unpublished WES test results[3] on riprap protection below energy dissipators indicates the following average diameter (D_{50}) stone size should result in adequate erosion protection.

$$D_{50} = D\left(\frac{V}{\sqrt{gD}}\right)^3$$

where

D_{50} = the minimum average size of stone, ft, whereby 50 percent by weight of the graded mixture is larger than D_{50} size

D = depth of flow in outlet channel, ft
V = average velocity in outlet channel, ft
g = gravitational acceleration, ft/sec^2

10. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets; Hydraulic Laboratory Model Investigation, by J. P. Bohan. Research Report H-70-2, Vicksburg, Miss., January 1970.

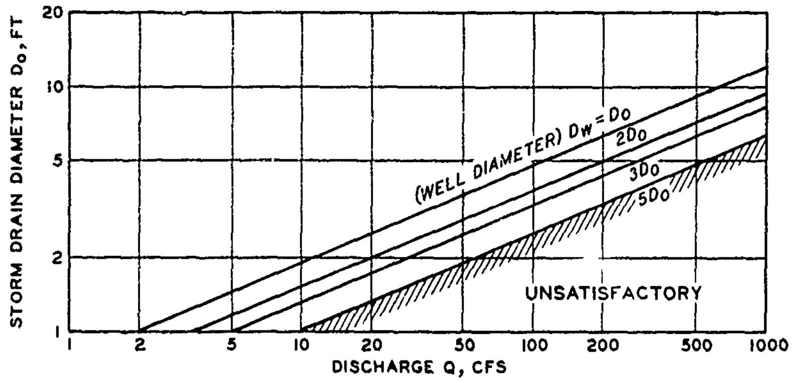
(2) _________, Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets, by B. P. Fletcher and J. L. Grace, Jr. Miscellaneous Paper H-72-5, Vicksburg, Miss., May 1972.

(3) _________, Evaluation of Three Energy Dissipators for Storm-Drain Outlets; Hydraulic Laboratory Investigation, by J. L. Grace, Jr., and G. A. Pickering. Research Report H-71-1, Vicksburg, Miss., April 1971.

(4) _________, Impact-Type Energy Dissipator for Storm-Drainage Outfalls Stilling Well Design; Hydraulic Model Investigation, by J. L. Grace, Jr. Technical Report No. 2-620, Vicksburg, Miss., March 1963.

(5) Beichley, G. L., Progress Report No. XIII - Research Study on Stilling Basins, Energy Dissipators and Associated Appurtenances - Section 14, Modification of Section 6 (Stilling Basin for Pipe or Open Channel Outlets - Basin VI. Report No HYD-572, Hydraulics Branch, Division of Research, U. S. Bureau of Reclamation, Denver, Colo., June 1969.

(6) Blaisdell, F. W., The SAF Stilling Basin. Agricultural Handbook No. 156, Agricultural Research Service and St. Anthony Falls Laboratory, University of Minnesota, Minneapolis, Minn., April 1959.



BASIC EQUATION

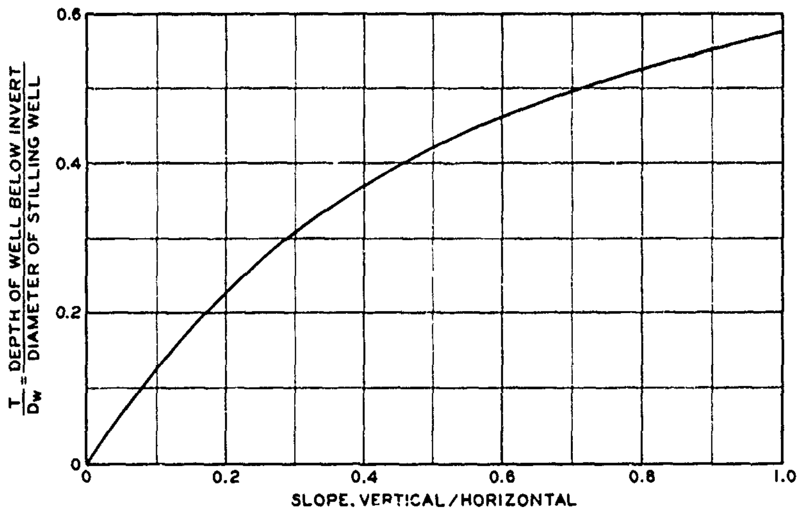
$$\frac{D_W}{D_O} = 0.53\left(\frac{Q}{D_O^{2.5}}\right) \text{ FOR } \frac{Q}{D_O^{2.5}} \le 10$$

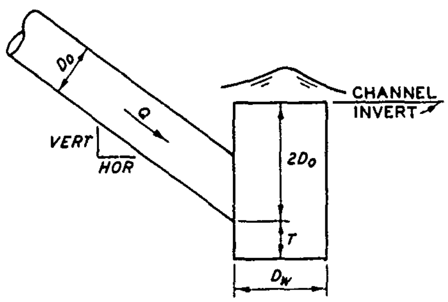
WHERE·

D_W = STILLING WELL DIAMETER, FT

D_O = DRAIN DIAMETER, FT

Q = DESIGN DISCHARGE, CFS



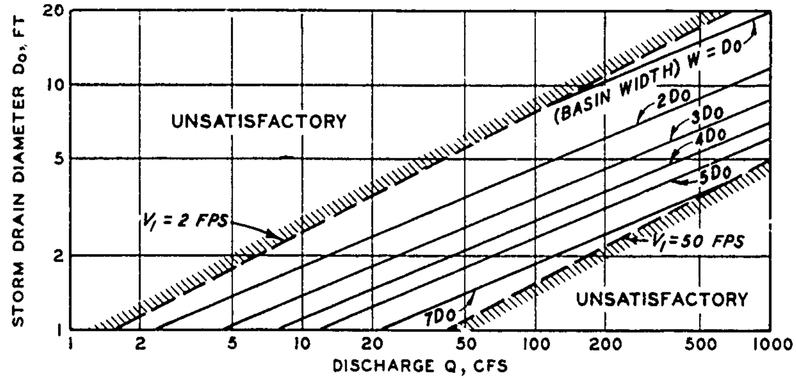


ELEVATION

STORM DRAIN OUTLETS
ENERGY DISSAPATORS
STILLING WELL

HYDRAULIC DESIGN CHART 722-1

WES 7-73

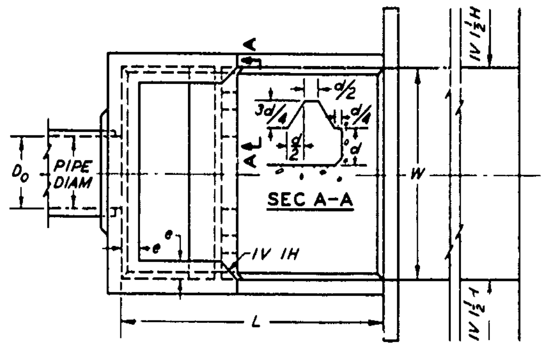


BASIC EQUATION

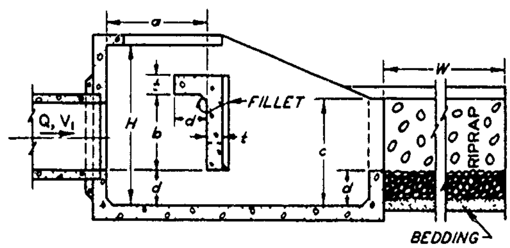
$$\frac{W}{D_0} = 1.3\left(\frac{Q}{D_0^{2.5}}\right) \quad \text{FOR} \quad \frac{Q}{D_0^{2.5}} \leq 21$$

WHERE.

- W = BASIN WIDTH, FT
- D_0 = DRAIN DIAMETER, FT
- Q = DESIGN DISCHARGE, CFS
- V_1 = PIPE VELOCITY, FPS



PLAN



SECTION

STILLING BASIN DESIGN

$H = \frac{3}{4}(W)$ $\quad c = \frac{1}{2}(W)$

$L = \frac{4}{3}(W)$ $\quad d = \frac{1}{6}(W)$

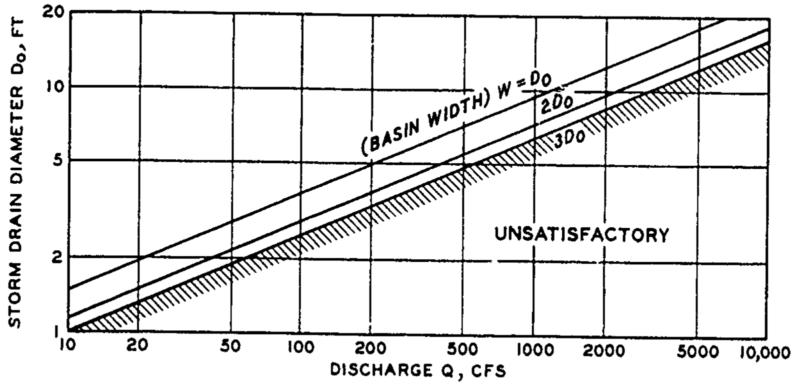
$a = \frac{1}{2}(W)$ $\quad e = \frac{1}{12}(W)$

$b = \frac{3}{8}(W)$ $\quad t = \frac{1}{12}(W)$, SUGGESTED MINIMUM

STORM DRAIN OUTLETS
ENERGY DISSAPATORS
IMPACT BASIN

HYDRAULIC DESIGN CHART 722-2

WES 7-73



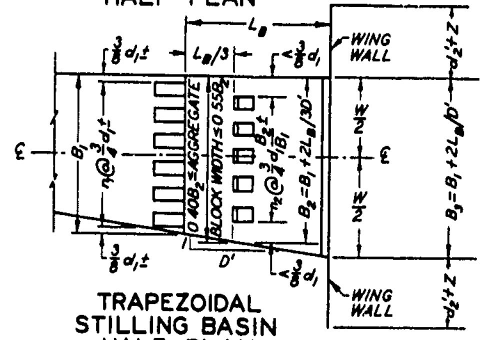
BASIC EQUATION

$$\frac{W}{D_0} = 0.3\left(\frac{Q}{D_0^{2.5}}\right) \text{ FOR } \frac{Q}{D_0^{2.5}} \le 9.5$$

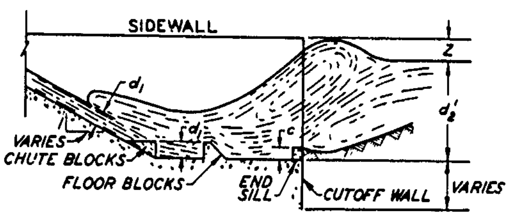
WHERE:

- W = END SILL LENGTH, FT
- D_0 = DRAIN DIAMETER, FT
- Q = DESIGN DISCHARGE, CFS

RECTANGULAR STILLING BASIN HALF-PLAN



TRAPEZOIDAL STILLING BASIN HALF-PLAN



CENTER-LINE SECTION

DESIGN EQUATIONS

$$F = \frac{V_1^2}{g d_1} \quad (1)$$

$$d_2 = \frac{d_1}{2}\left(-1 + \sqrt{8F+1}\right) \quad (2)$$

F = 3 TO 30	$d_2' = (1.10 - F/120)\, d_2$	(3a)
F = 30 TO 120	$d_2' = 0.85 d_2$	(3b)
F = 120 TO 300	$d_2' = (1.00 - F/800)\, d_2$	(3c)

$$L_B = \frac{4.5 d_2}{F^{0.38}} \quad (4)$$

$$Z = \frac{d_2}{3} \quad (5)$$

$$c = 0.07 d_2 \quad (6)$$

STORM DRAIN OUTLETS
ENERGY DISSAPATORS
STILLING BASIN

HYDRAULIC DESIGN CHART 722-3

WES 7-73

HYDRAULIC DESIGN CRITERIA

SHEET 722-4 TO 722-7

STORM DRAIN OUTLETS

RIPRAP ENERGY DISSIPATORS

1. Purpose. Criteria for the hydraulic design of fixed energy dissipating structures for storm drain outlets are presented in Hydraulic Design Charts (HDC's) 722-1 to 722-3. Under some conditions adequate energy dissipation can be accomplished more economically using riprap as an alternate to fixed structures. HDC's 722-4 to 722-5 present three basic riprap energy dissipator designs developed at WES.[1,2]

2. Scour Holes. Scour holes at storm drain exit portals effectively dissipate flow energy and reduce downstream erosion. However, uncontrolled scour holes can undermine the storm drain with subsequent structural failure. Basic laboratory tests were conducted at WES[1] during the period 1963-1969 to investigate scour hole development and erosion protection in cohesionless material downstream from storm drain exit portals. These tests showed that the length, width, depth, and volume of the scour hole could be related in terms of the storm drain diameter D_o in feet, the discharge Q in cfs, and the flow duration t in minutes. The tailwater depth TW in feet over the storm drain invert was also found to be important. The following set of design equations[2] describes the basic scour hole dimensions for two controlling tailwater conditions.

$$\frac{L_{sm}}{D_o} = C\left[\left(\frac{Q}{D_o^{2.5}}\right)^{0.71}\left(t^{0.125}\right)\right] \qquad (1)$$

$$\frac{D_{sm}}{D_o} = C\left[\left(\frac{Q}{D_o^{2.5}}\right)^{0.375}\left(t^{0.10}\right)\right] \qquad (2)$$

$$\frac{W_{sm}}{D_o} = C\left[\left(\frac{Q}{D_o^{2.5}}\right)^{0.915}\left(t^{0.15}\right)\right] \qquad (3)$$

$$\frac{V_s}{D_o^3} = C\left[\left(\frac{Q}{D_o^{2.5}}\right)^{2}\left(t^{0.375}\right)\right] \qquad (4)$$

where

L_{sm} = scour hole length, ft
D_{sm} = depth of maximum scour, ft
W_{sm} = half the width of the hole at the location of maximum scour, ft
V_s = volume of material removed from scour hole, ft^3

Empirically determined values of C in the equations above for the two controlling tailwater conditions are:

$\frac{TW}{D_o}$	Equation No. 1	2	3	4
>0.5	4.10	0.74	0.72	0.62
≤0.5	2.40	0.80	1.00	0.73

3. HDC 722-4 shows dimensionless scour hole profiles and cross sections for the two limiting tailwater conditions.

4. Horizontal Riprap Blanket. HDC 722-5 shows the recommended length L_{sp} and geometry of the horizontal riprap blanket protection required for satisfactory dissipation of the energy of the design outflow from a storm drain. (The required D_{50} riprap size can be estimated using HDC 722-7.)

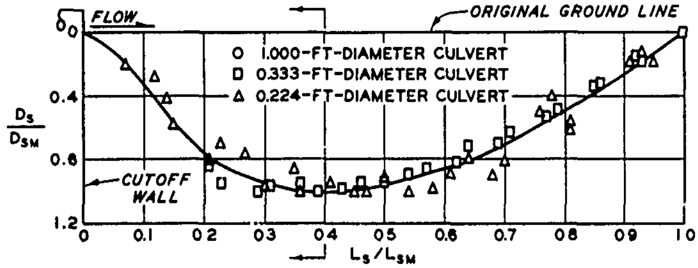
5. Preformed Scour Holes. Laboratory studies have shown that satisfactory energy dissipation of storm drain outflow occurs in riprap-lined, preformed scour holes of nominal size. HDC 722-6 shows the recommended design for preformed scour holes 0.5 and $1.0D_o$ deep. The D_{50} minimum stone size required for each scour hole depth can be estimated using HDC 722-7.

5. Application. Study of the basic test data indicates that the resulting design criteria are generally applicable to both circular and rectangular conduits flowing full or partly full. For rectangular conduits the conduit width is used in place of the diameter D_o of the circular conduits.

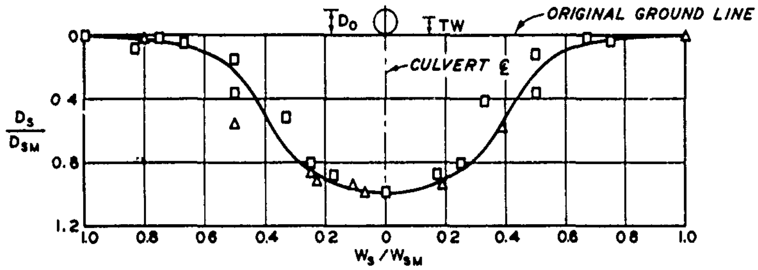
6. References.

(1) U. S. Army Engineer Waterways Experiment Station, CE, Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets; Hydraulic Model Investigation, by J. P. Bohan. Research Report H-70-2, Vicksburg, Miss., January 1970.

(2) _________, Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets, by B. P. Fletcher and J. L. Grace, Jr., Miscellaneous Paper H-72-5, Vicksburg, Miss., May 1972.

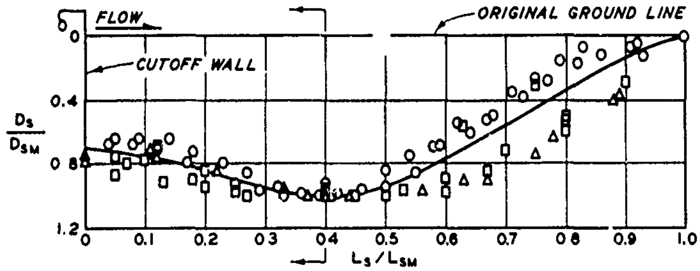


DIMENSIONLESS CENTER-LINE PROFILE

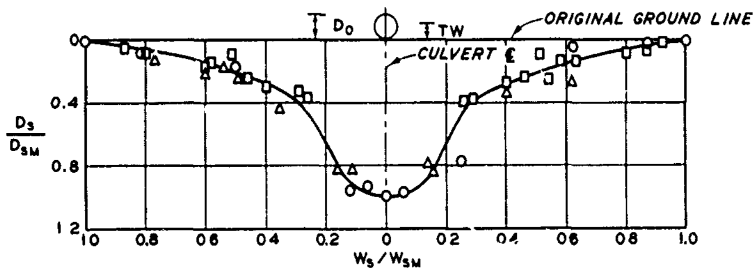


DIMENSIONLESS CROSS SECTION AT $0.4L_{SM}$

$TW > 0.5D_O$



DIMENSIONLESS CENTER-LINE PROFILE



DIMENSIONLESS CROSS SECTION AT $0.4L_{SM}$

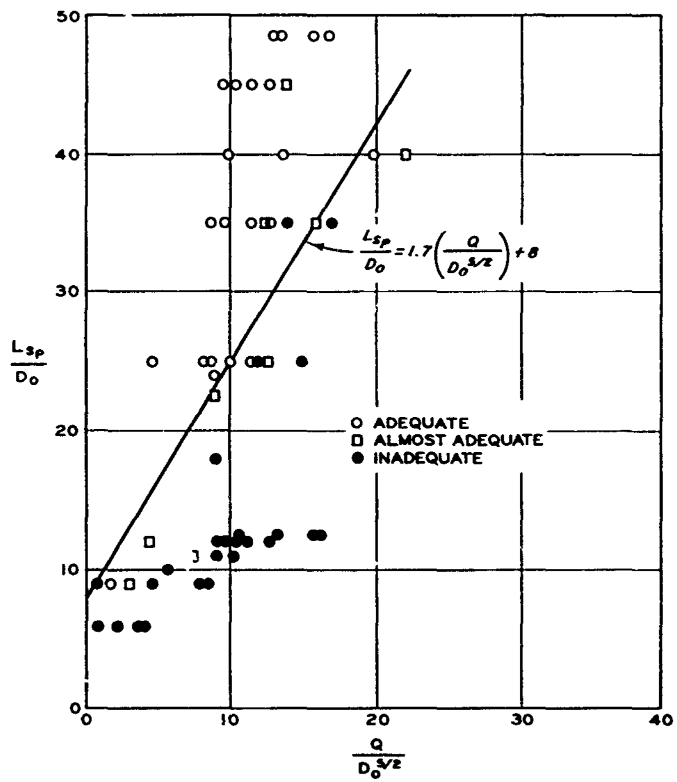
$TW \leq 0.5D_O$

NOTE L_S = DISTANCE FROM OUTLET TO D_S, FT
L_{SM} = DISTANCE FROM OUTLET TO END OF SCOUR, FT
W_S = DISTANCE (R & L) FROM ℄ TO D_S AT $0.4L_{SM}$, FT
W_{SM} = DISTANCE (R & L) FROM ℄ TO $0.0D_S$ AT $0.4L_{SM}$, FT
D_O = DIAMETER OR WIDTH OF STORM DRAIN, FT
TW = TAILWATER DEPTH ABOVE DRAIN INVERT, FT
D_S = DEPTH OF SCOUR, FT
D_{SM} = MAXIMUM SCOUR DEPTH, FT

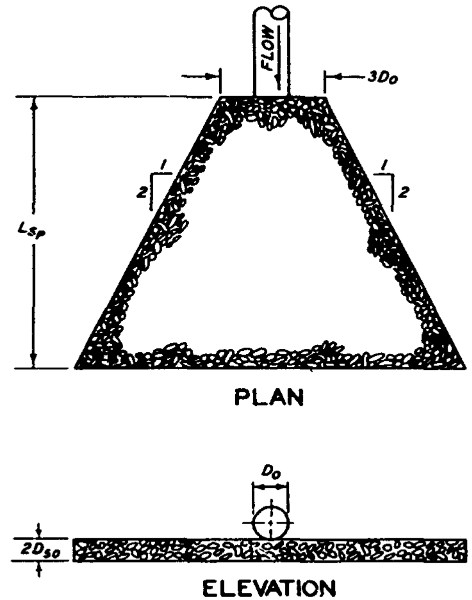
STORM DRAIN OUTLETS
SCOUR HOLE GEOMETRY
$TW > 0.5D_O$ AND $\leq 0.5D_O$

HYDRAULIC DESIGN CHART 722-4

WES 7-73



NOTE: D_o - DIAMETER OR WIDTH OF STORM DRAIN, FT
Q - STORM DRAIN DISCHARGE, CFS
L_{sp} - HORIZONTAL LENGTH OF BLANKET, FT

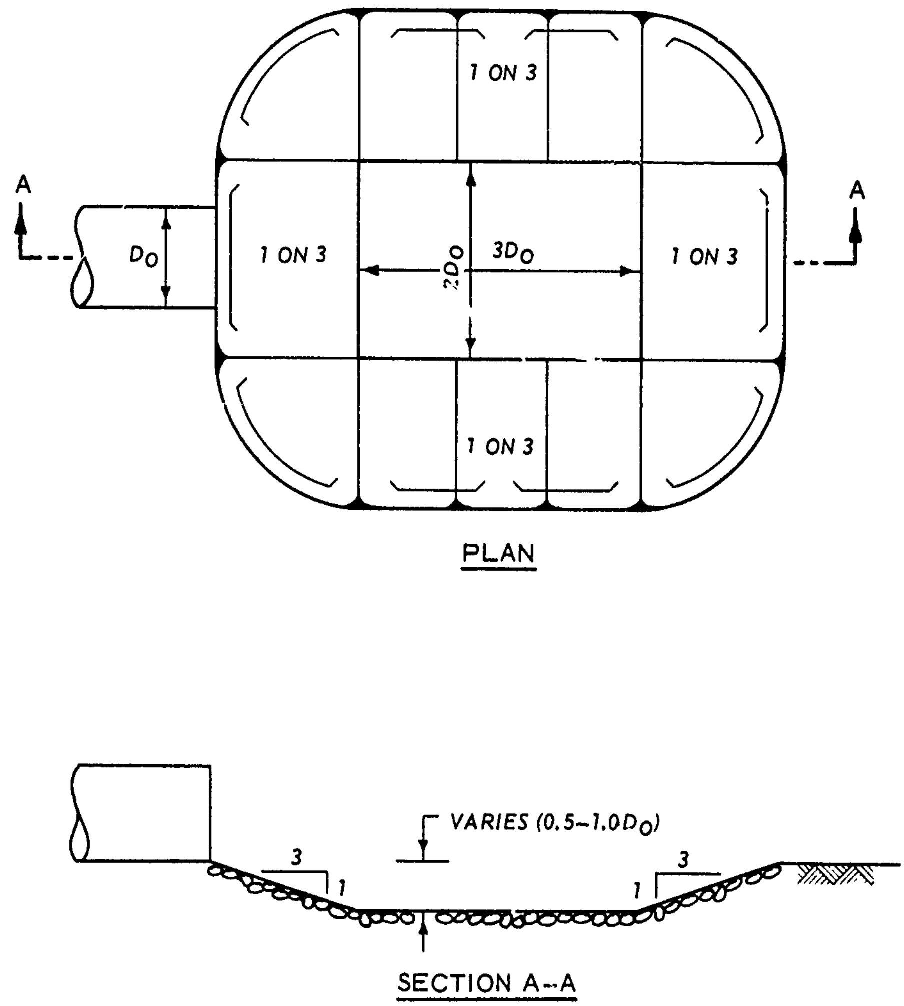


NOTE: D_{50} - MINIMUM AVERAGE SIZE OF STONE, FT

STORM DRAIN OUTLETS
RIPRAP ENERGY DISSAPATORS HORIZONTAL BLANKET LENGTH OF STONE PROTECTION

HYDRAULIC DESIGN CHART 722-5

WES 7-73



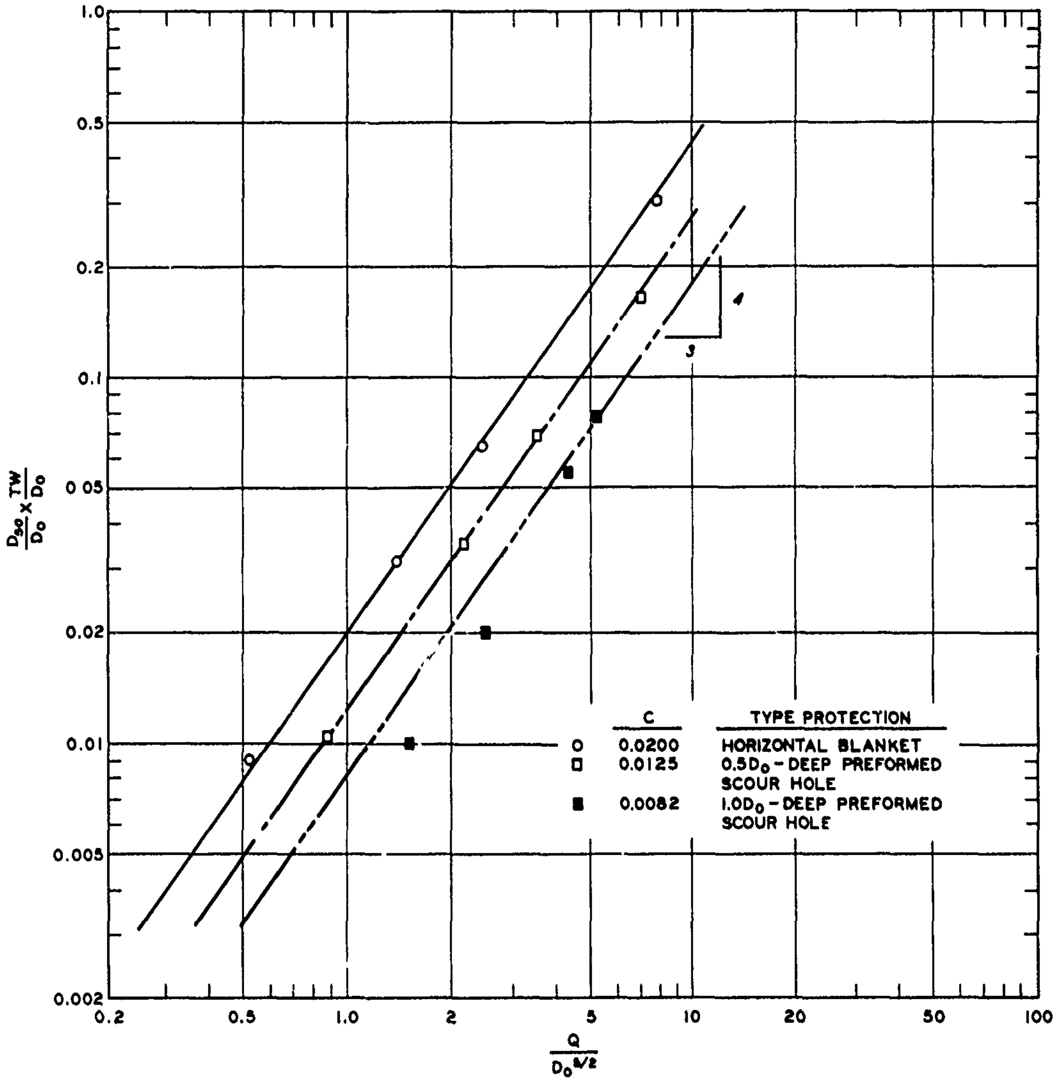
NOTE: D_O = DIAMETER OR WIDTH OF STORM DRAIN, FT

STORM DRAIN OUTLETS

RIPRAP ENERGY DISSAPATORS
PREFORMED SCOUR HOLE GEOMETRY

HYDRAULIC DESIGN CHART 722-6

WES 7-73



BASIC EQUATION

$$\frac{D_{50}}{D_o} = C \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}} \right)^{4/3}$$

WHERE:

D_{50} = MINIMUM AVERAGE SIZE OF STONE, FT
D_o = DIAMETER OR WIDTH OF STORM DRAIN, FT
Q = STORM DRAIN DISCHARGE, CFS
TW = TAILWATER DEPTH ABOVE DRAIN INVERT, FT

STORM DRAIN OUTLETS
RIPRAP ENERGY DISSAPATORS
D_{50} STONE SIZE

HYDRAULIC DESIGN CHART 722-7

WES 7-73

HYDRAULIC DESIGN CRITERIA

SHEET 733-1

SURGE TANKS

THIN PLATE ORIFICES

HEAD LOSSES

1. Thin plate orifices are often used in surge tank risers to restrict the flow during load-on and load-off operations. Computation of the head losses through these orifices is of interest in the design of surge tanks.

2. A number of experiments have been made on head losses through orifices in straight pipe. When an orifice is placed in a surge tank riser close to the penstock tee, the energy loss of flow entering or leaving the riser is affected by the orifice flow. Indri's[2] extensive study of orifices in branches has made available new data on head loss coefficients considered to be applicable to surge tank problems. The pipe used in this study was 9 cm (3.54 in.) in diameter. The orifice plates were located in the branches 125 mm (4.92 in.) from the center line of the main pipe. The test results indicate that the combined tee and orifice loss coefficients were independent of Reynolds number for $R_e > 3 \times 10^4$.

3. HDC 733-1 presents a head loss coefficient curve for thin plate orifices in tees. The head loss coefficient is based on the combined tee and orifice head loss. Indri's data shown in this chart indicate that a single curve is applicable to load on-load off turbine conditions. Also shown in this chart are head loss coefficient curves by Weisbach[3] and Marchetti[1] for thin plate orifices in straight pipe. These curves indicate that the location of the orifice with respect to other disturbances affects the head loss.

4. The data in HDC 733-1 are based on the equation:

$$H_L = K_o \frac{V^2}{2g}$$

where

H_L = head loss across the orifice or orifice and tee, ft
K_o = head loss coefficient
V = velocity in riser, ft per sec

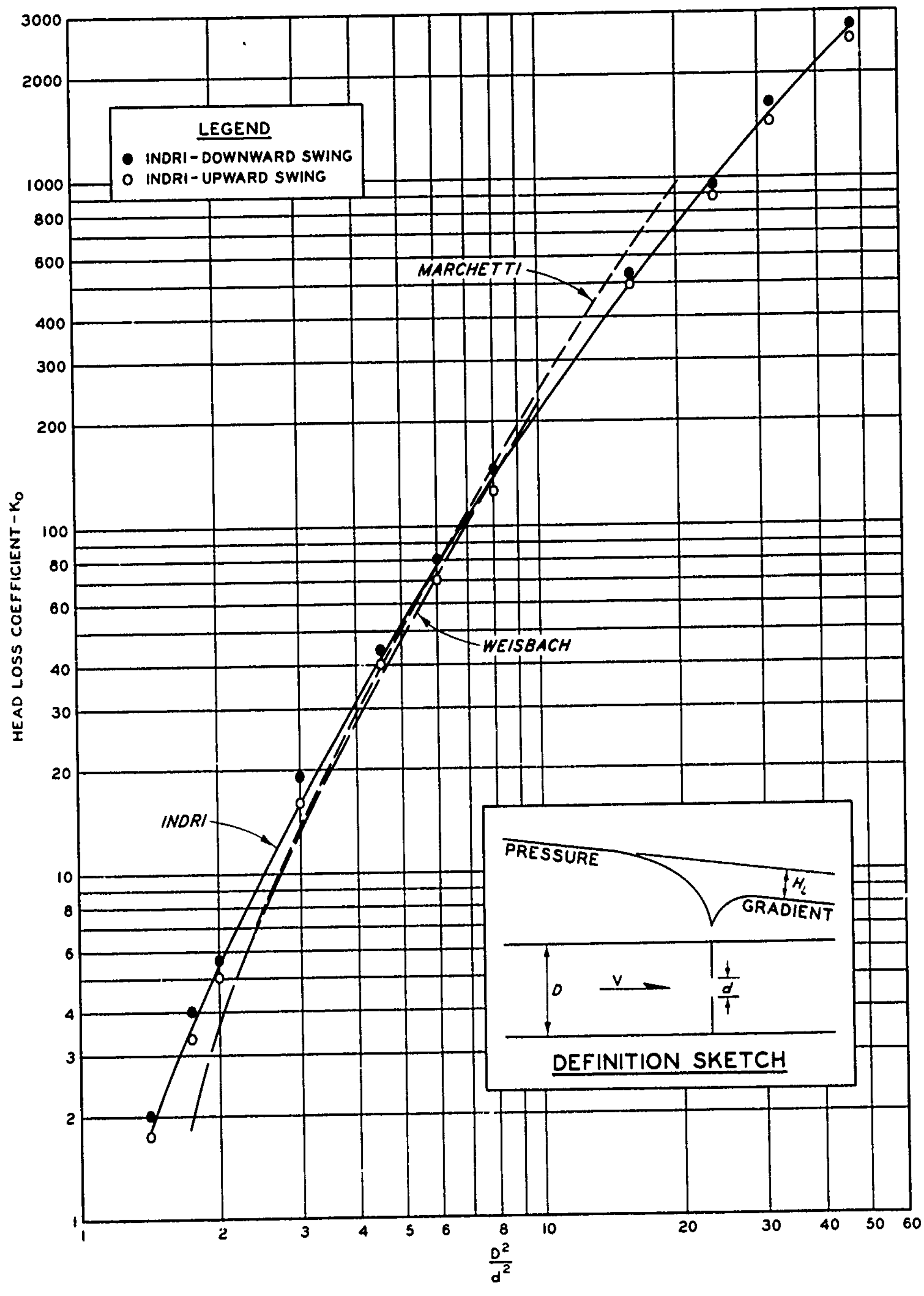
The head loss coefficient is plotted as a function of the ratio of the square of the riser diameter D to the square of the orifice diameter d. A sketch of an orifice in a straight pipe is included in the chart for purposes of defining the terms involved.

5. References.

(1) Caric, D. M., "Tehnicka hydraulika." Gradevenska, Knjiga, Belgrad (1952).

(2) Indri, E., "Richerche sperimentali su modelli di strozzature per pozzi piezometrici (Experimental research on models of constrictions for surge tanks)." L'Energia Elettrica, vol 34, No. 6 (June 1957), pp 554-569. Translation by Jan C. Van Tienhoven, for U. S. Army Engineer Waterways Experiment Station, CE, Translation No. 60-3, Vicksburg, Miss., April 1960.

(3) Weisbach, J., Untersuchungen in den Gebieten der Mechanik und Hydraulik. Leipzig, 1945.



EQUATION

$$H_L = K_O \frac{V^2}{2g}$$

WHERE

H_L = HEAD LOSS ACROSS ORIFICE, FT

K_O = HEAD LOSS COEFFICIENT

V = VELOCITY IN PIPE, FT PER SEC

SURGE TANKS
THIN PLATE ORIFICES
HEAD LOSSES

HYDRAULIC DESIGN CHART 733-1

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI

WES 10-61